SEWAGE AND INDUSTRIAL WASTES

(Formerly Sewage Works Journal)

JOURNAL of the FEDERATION OF SEWAGE AND INDUSTRIAL WASTES ASSOCIATIONS

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FEBRUARY, 1956—VOL. 28, No. 2

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IN CONJUNCTION WITH

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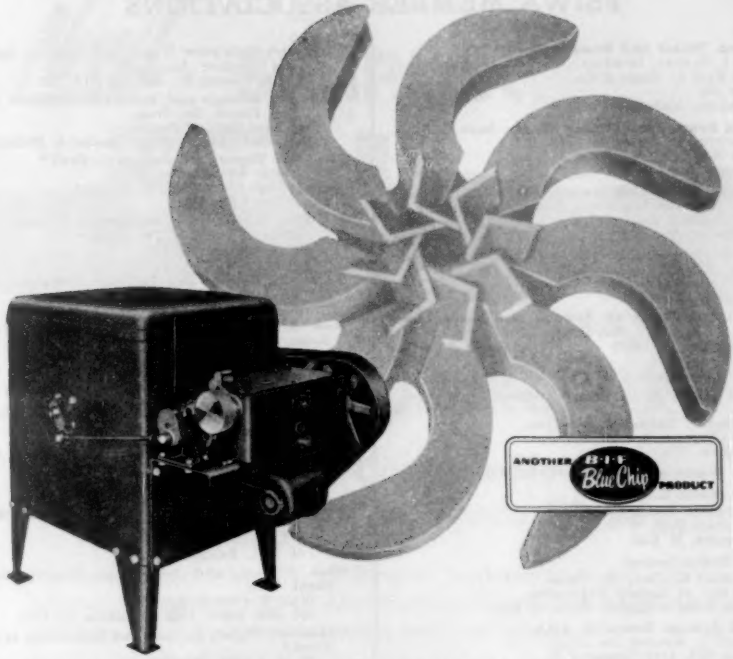

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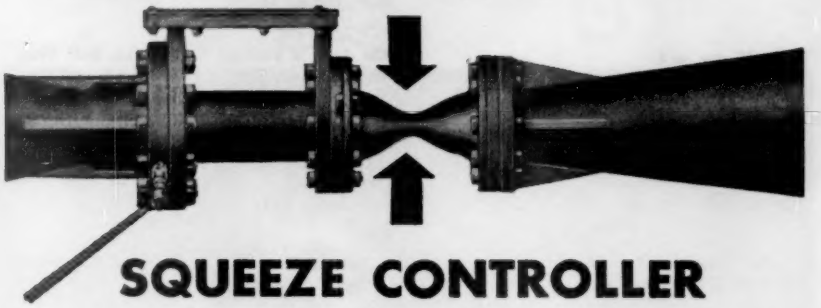

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New Jersey Sewage and Industrial Wastes Assn.	Hotel Traymore Atlantic City, N. J.	Mar. 14–16, 1956
Arkansas Water and Sewage Conference (*Silver Anniversary*)	Marion Hotel Little Rock, Ark.	Mar. 19–21, 1956
Kansas Sewage and Industrial Wastes Assn.	Jayhawk Hotel Topeka, Kansas	Apr. 4–6, 1956
Montana Sewage and Industrial Wastes Assn.	Murray Hotel Livingston, Montana	Apr. 5, 1956
Arizona Sewage and Industrial Wastes Assn.	Buena Vista Hotel Safford, Arizona	Apr. 5–7, 1956
Virginia Industrial Wastes and Sewage Works Assn.	Natural Bridge Hotel Natural Bridge, Va.	Apr. 19–20, 1956
California Sewage and Industrial Wastes Assn.	Santa Rosa Hotel Santa Rosa, Calif.	Apr. 25–28, 1956
Maryland-Delaware Water and Sewage Assn.	Willard Hotel Washington, D. C.	May 10–11, 1956
Michigan Sewage and Industrial Wastes Assn.	Whitcomb Hotel St. Joseph, Mich.	May 14–16, 1956
Alabama Water and Sewage Assn.	Alabama Polytechnic Auburn, Ala.	June 4–6, 1956
Ohio Sewage and Industrial Wastes Treatment Conference	Hotel Carter Cleveland, Ohio	June 13–15, 1956
Institute of Sewage Purification	Harrogate, England	June 18–22, 1956
Central States Sewage and Industrial Wastes Assn.	Northland Hotel Green Bay, Wis.	June 20–22, 1956

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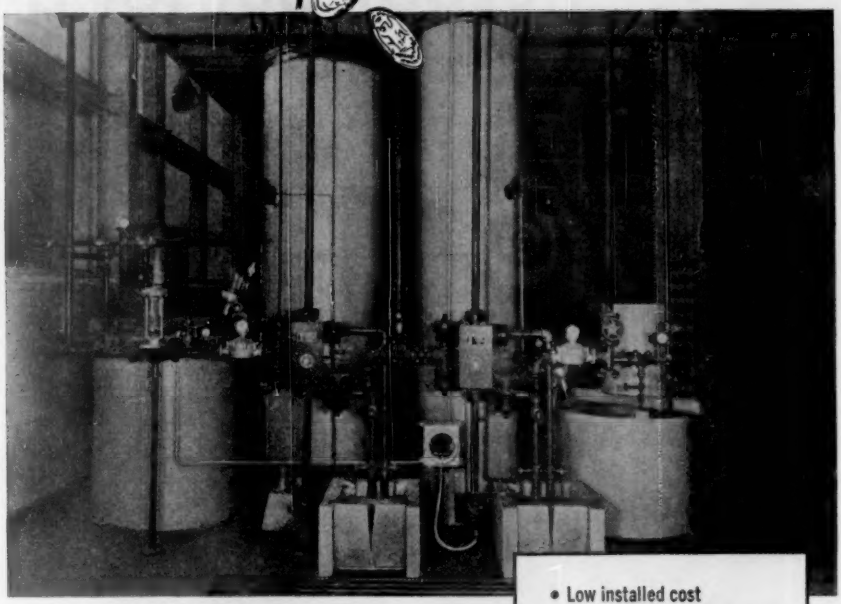

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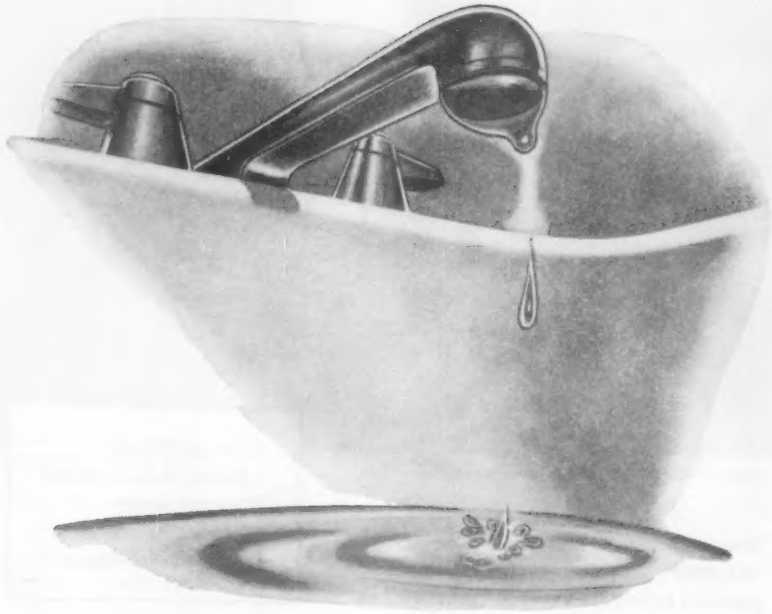

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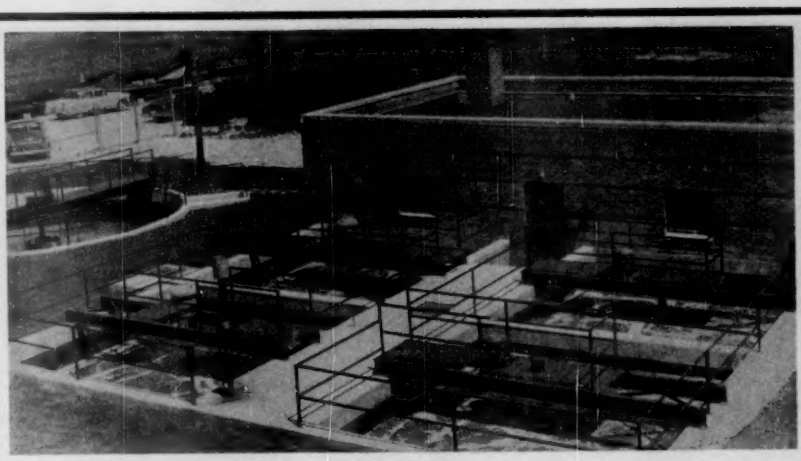

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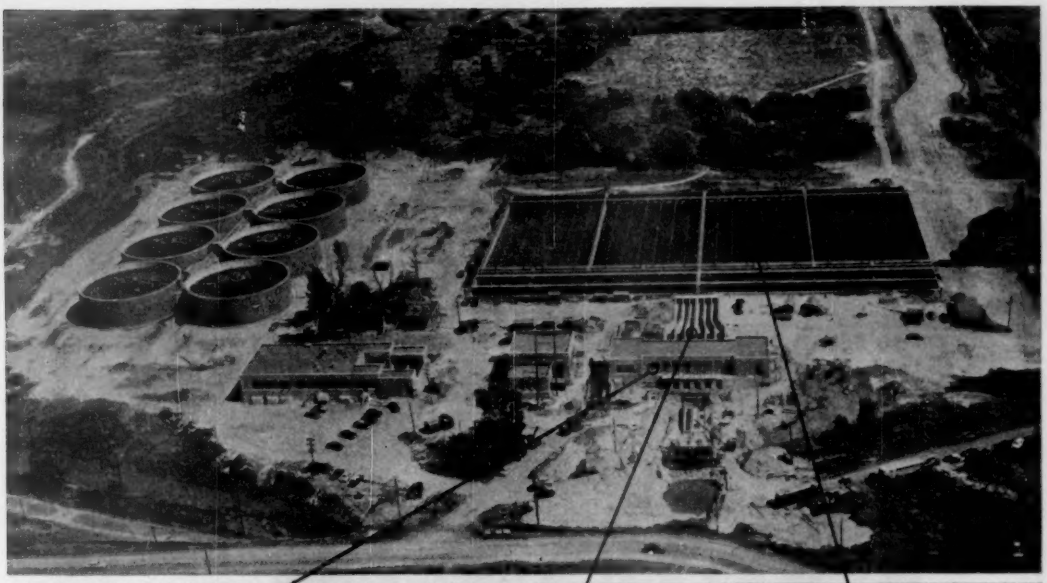

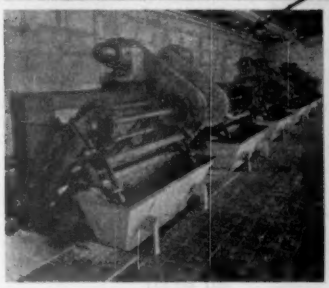

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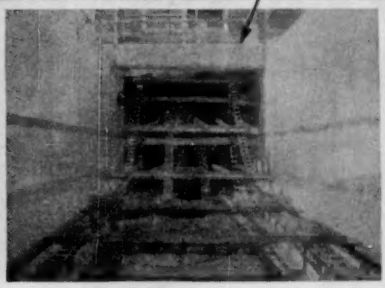

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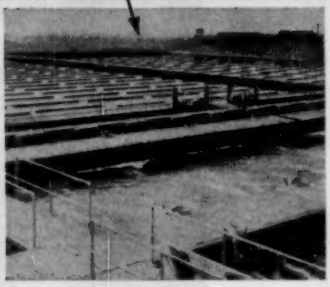

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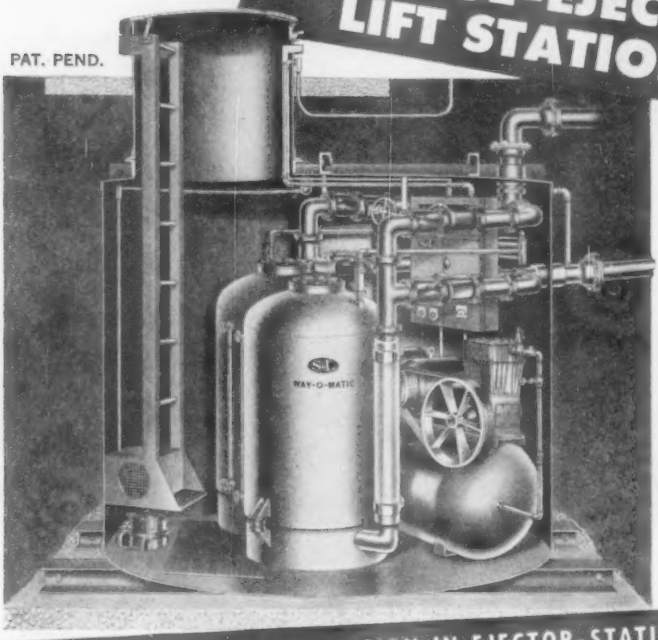

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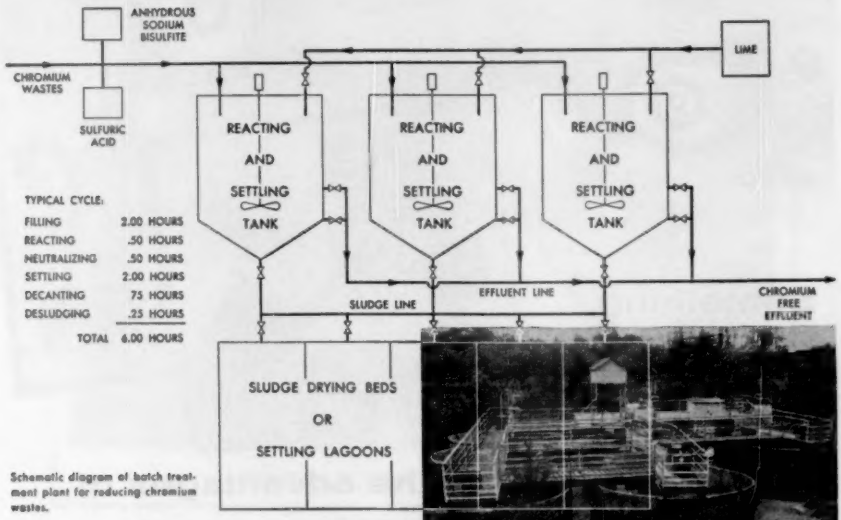

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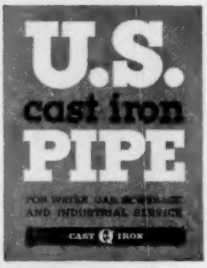

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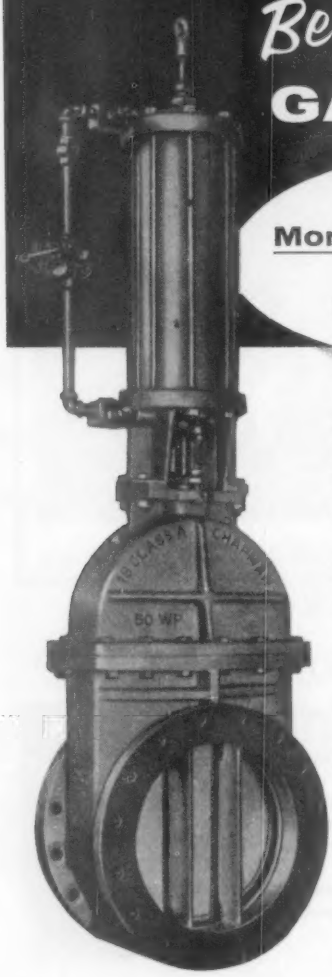

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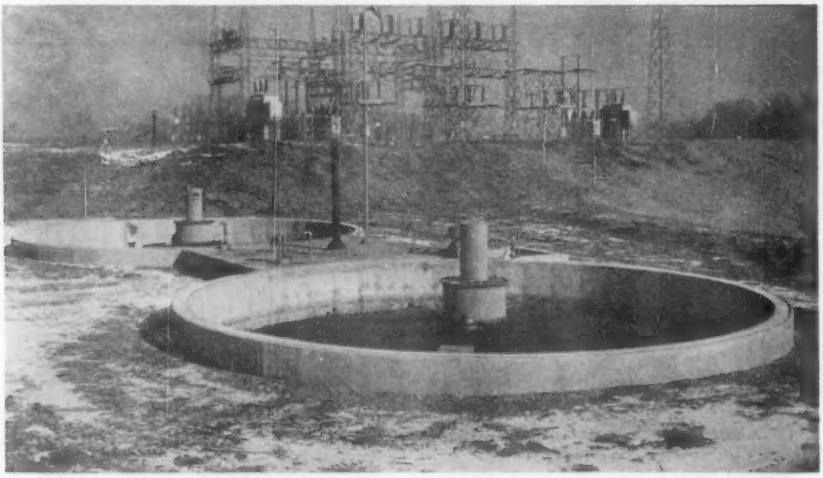

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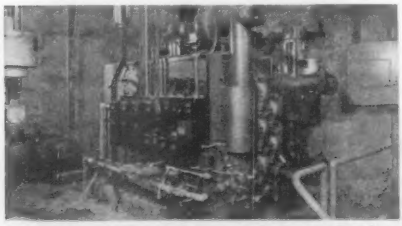

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SEWAGE AND INDUSTRIAL WASTES

Journal of the Federation of Sewage and Industrial Wastes Associations

Volume 28 **February, 1956** **Number 2**

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Ralph E. Fuhrman, *Editor*, 4435 Wisconsin Ave., N.W., Washington 16, D. C.
Dr. F. W. Mohlman, *Advisory Editor* Donald P. Schiesswohl, *Assistant Editor*
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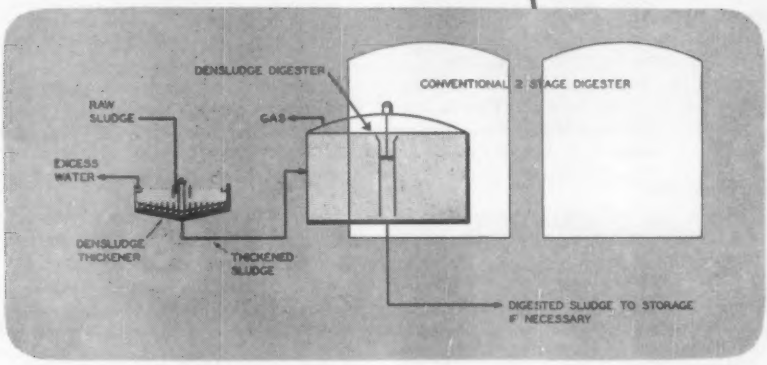

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Sewage Works

PIPE LINE TRANSPORTATION AND DISPOSAL OF DIGESTED SLUDGE *

By John J. Wirts

Superintendent, Easterly Sewage Treatment Plant, Cleveland, Ohio

Man's quest for an ultimate method of disposal for the by-products of civilization has been a long and sometimes discouraging search.

This paper deals with the handling of principally organic material in connection with modern methods of sewage treatment. Mass movement of solids by pipe line is discussed as a useful tool that has interesting possibilities. The pulping of other organic wastes, such as garbage, rubbish, waste paper, and the like, could well be made a subject for further study. Certain of the industrial wastes, such as alkaline sulfides, spent caustic and pickling liquors have been found to be compatible with sludge solids and should be included.

The basic requirement in any mass transportation system is that at least two or more well established locations be connected by some media of transport. It is relatively unimportant whether the means of conveyance be railroads, belt conveyors, pipe line or barging. In Ohio, the largest commercial strip mine coal operation in the world is located about 100 miles south of Lake Erie. The south shore of Lake Erie with its low cost power, abundant fresh water supply, and land at reasonable cost has attracted industry.

Digested sludge, along with compatible industrial wastes, would find ultimate disposal at the strip mines. The economics of the dual pipe line system indicate advantages that would not exist in the one way operation of either force main. Hence the inclusion of coal is important because it makes the capital risk a more attractive investment.

Coal Pumping

In June 1955, the Pittsburgh Consolidation Coal Co. successfully negotiated a contract with the Cleveland Electric Illuminating Co. for the delivery by pipe line of 18,000,000 tons of coal over a 15 year period. The pipe line will connect the Georgetown, Ohio, mining area of the coal company with the Eastlake Generating Station, situated on the shore of Lake Erie, a distance of 108 miles.

The approximate basis of design of the 108 mile force main includes three reciprocating pump stations, rated at 1,000 p.s.i. while pumping a 50 per cent mixture of coal and water through 10.37 in. inside diameter extra heavy steel pipe. The pumps will discharge 125 tons of minus ⅛-in. coal per hour (1,200,000 tons per year) at a velocity of 5 ft. per second. Separation of the water carrier and necessary coal drying will be done at the Eastlake site. The design does not include a return water line. The project will cost 8 to 10 million dollars. For competitive economic reasons the total pumping cost cannot exceed approximately $0.02 per ton mile. Depreciation of the investment over a 15 year period would

* Presented at 28th Annual Meeting, Federation of Sewage and Industrial Wastes Assns.; Atlantic City, N. J.; October 10–13, 1955.

be included in the total cost of $0.02 per ton mile. Over two years of experimental pumping preceded the signing of the contract. In this work, 6,000 ft. of 12¾ in. I. D. pipe was laid in the hill country at Cadiz, Ohio.

Historical Background

Recent studies by Dougherty (1), Durand (2), and Pursglove (3) have furnished accurate data, making it possible for industry to consider pipe line transportation of solids. Although the first patent, broadly covering a method of pumping coal and water, was issued to W. C. Andrews about 1895, the economical side of the method had never been explored thoroughly. It is for this reason and the fact that the field of application presents economic problems somewhat foreign to industry, that limited use has been made of the process. Many short lines do exist for the hydraulic transportation of solids, but they are usually limited to interplant process conditions.

There is an interesting similarity between the history of coal pumping and the history of digested sludge pumping.

Birmingham, Eng.

The earliest known case is that of Birmingham, Eng., where a 2.5-mile length of 9-in. cast iron main was constructed in 1902 and operated by steam driven ram pumps. This main was extended in 1907 to four miles in length, and a parallel pipe 12 in. in diameter was added at a later date.

Separate sludge digestion was commenced at Birmingham in about 1909, but previously the sedimentation tanks had been operating as septic tanks, so that the pumping has been carried out continuously with digested material. Commenting on this pumping operation, John D. Watson (4), then Engineer to the Birmingham Drainage Board, stated "The task looked more formidable than it turned out to be, the sludge in its fermented state having been found to be more mobile than expected, a condition probably due to the innumerable gas-bubbles between particles acting upon it like so many ball bearings."

In contrast with this experience, there has been trouble at Manchester, Eng. with a main conveying undigested sludge in a pipe for a distance of only one mile, due to caking with grease. Many similar instances have occurred both in England and the United States.

At Birmingham the 12-in. main now operates with a friction loss of 2.6 times that of water with sludge having 8.5 to 10.25 per cent solids. The Birmingham material is an industrial sludge from a completely combined sewerage system. The raw sludge or-

TABLE I.—Force Mains Used for Sludge Transportation

Location	Length (miles)	Diameter (in.)	Total Head (ft.)	Sludge Type	Dry Solids (%)
Mogden, Eng.	7	12	142	Digested	4.0–5.0
Birmingham, Eng.	4	9 and 12	2.6×water	Digested	8.5–10
Chicago, Ill.	17	14	210	Raw	1.0–2.0
	5	12	170	Raw	2.0–4.0
Cleveland, Ohio	13	12	391	Raw	3.0–4.0
Philadelphia, Pa.	5	8	225	Raw	3.0–4.0
Columbus, Ohio[1]	5	—	—	—	4.0–5.0
The Hague, Netherlands[2]	7	8	—	Digested	4.0–5.0
Los Angeles, Calif.[3]	7.5	24	—	Digested	3.73
Chicago, Ill.[4]	75–100	—	—	Raw	2.0

[1] Connecting two treatment plants.
[2] For disposal in North Sea.
[3] For disposal in Pacific Ocean.
[4] To strip mine area.

ganic content is about 60 per cent, as compared with a normal 75 per cent volatile content.

Since the early beginning of sludge force main pumping at the start of the 20th century, a considerable number of mains have been placed in operation. Table I, presents five completed installations and four proposed installations. With the exception of the treatment facilities at the Hague (5), Los Angeles, Calif. and Chicago, Ill., all of the mains are interprocess and relatively short in length. It has sometimes been economical, especially in the larger cities, to develop sludge processing sites remote from the sewage treatment plant. In these cases the location, space limitation, land value and foundation condition warrant the pumping of sludge several miles to a better suited area. Low pressure force mains, operating at pressures less than 200 p.s.i. have been used to convey solids in concentrations less than 10 per cent. High pressure pumping in the range of 800 ft. of head (four single stage pumps in series) has not been attempted.

Chicago, Ill.

Mohlman (6) in 1935 discussed in detail the results of comprehensive pilot plant work used as a basis of design for the most elaborate sludge processing plant in the world. All known methods were carefully evaluated, with the exception of sludge transportation to low cost disposal areas. Mohlman concluded: "The development of flash drying and incineration at the Sanitary District of Chicago followed extensive investigations of other methods of disposal of sewage solids, including Imhoff tanks, heated separate digestion tanks, sand drying beds, vacuum filters and various types of dewatering machines." As a result, a contract was subsequently let at the Calumet Treatment Works for a similar dewatering and incinerating plant for handling the settled and activated sludge from this sewage treatment plant, amounting to 40 tons of dry solids per day.

Twenty years later, Langdon Pearse (7) reported that at Calumet, about one-half the sludge tonnage received is heat dried and sold, the balance is being lagooned. At the West-Southwest Works, 183 acres of lagoons have received an average of 87,000 tons of dry solids per year, which nearly equals the best year of fertilizer production, which was 92,000 tons in 1953. The sludge processing program as outlined by Mohlman 20 years ago has been developed to the extent of about one-half the total requirement. The Sanitary District of Chicago has transported a major portion of the raw sludge production between widely separated treatment plants. If lagoons are considered a step in the treatment process, their experience can be classed as interprocess pumping.

Recently the Sanitary District of Chicago has become interested in long distance pumping. Dundas (8) reports that he has had the same pumping problem under consideration for the past three months and will soon begin a study survey for pumping activated sludge a distance of 75 to 100 miles for ultimate disposal in abandoned strip mines Southwest of Chicago. Extremely difficult problems are not contemplated and it is believed that the operation will prove economical.

Cleveland, Ohio

The Cleveland program of pollution control parallels in time the Chicago construction. During the early 1930's plant-scale demonstration units on digestion, vacuum filtration and incineration were thoroughly explored. Perhaps the earliest and best appraisal of sludge dewatering costs was estimated in 1934. Jones (9) stated that "Operation and maintenance cost, exclusive of interest and depreciation, is estimated at $5.20 per ton dry solids. The total annual cost, $12.50 per ton dry

solids." Considerable professional criticism was received by Jones at that time, because of the relatively high costs. However these early estimates were later shown to be amazingly accurate and current costs are more than twice the original estimates.

The sludge production record at Cleveland is similar in many respects to that of Chicago. About one-half of the digested solids are dewatered and incinerated. The balance is pumped to lagoons, or in the case of supernatant, pumped to the Cuyahoga River.

Although a great deal of time and money has been spent on sludge processing, Cleveland, like Chicago, has never intensively studied the transportation of solids to remote locations suitable for disposal. Many similar instances have occurred elsewhere. The required expansion of the Cleveland sludge processing units is being temporarily held up pending further study.

Experience at Cleveland (10) with interprocess pumping of sludge through a 12-in. force main, 13 miles long, has extended over a period of 17 years. During this time considerable experience has been gained in the knowledge of sludge pumping and improvements have been made in pump design.

New York City, N. Y.

Since the opening of the Wards Island Sewage Treatment Plant in 1937, the ultimate disposal of sludge has been by barging to sea. Three large sludge vessels travel 68 miles round trip to the ocean dumping grounds. Donaldson (11) estimated the operation and maintenance cost per ton of dry solids at $2.50 to $3.00 for the first six months of operation. The steady increase in the cost of labor and material experienced by New York City and other municipalities has resulted in a total cost (1953) of $9.80 per ton of dry solids, including interest and amortization. Operation costs for 1953, according to O'Leary (12) were $7.96 per ton, with interest and amortization amounting to $1.84 per ton of dry solids.

The wisdom exercised by Gould and Donaldson in the light of present experience, has resulted in the ultimate disposal of all the dry solids at a tonnage cost less than the Jones estimate made in 1934. Furthermore, New York City is the only major city, with the possible exception of Milwaukee, Wis., that has been able to handle all of the solids removed from the sewage during the past 17 years. New York City is, of course, continuously investing in treatment works enlargements and completely new projects as their pollution control program expands. For the purpose of soil conditioner, New York City, in cooperation with the Park Department, has proceeded with the construction of two short force mains for limited pumping of digester sludge. The Park Department will use the sludge as an admixture in making synthetic top-soil.

Mogden, Eng.

The handling of sludge at the Mogden sewage treatment facilities is similar, in many respects, to the problem at Cleveland. About 35,000 tons of raw sludge is produced annually. The point of difference is that by digesting the sludge at the Mogden treatment plant the solids are reduced 35 to 40 per cent prior to pumping. Secondary digestion and sludge drying beds, located at Perry Oaks, about 7 miles distant, are connected to Mogden by a 12-in. cast iron force main. Townend (4), commenting on this main, wrote: "This operation has been carried out without the slightest trouble at any time. The average pumping rate is about 550 (Imp.) g.p.m., giving a velocity of 1.87 f.p.s. in the 12-in. rising main, and occupies about 12 hr. each day. At this velocity, the total delivery head is about 142 ft., including a static lift above the pumps of 65 ft.; thus the total friction head is 77 ft., or 11 ft.

per mile. This friction loss for digested sludge of 96 to 97 per cent water content, works out at about 1.4 that for water under the same conditions."

The Hague, Netherlands

A unique approach to the sludge disposal problem has come from the Netherlands. Late in 1954, the Hague Government sent Ir. A. Mast, Chief Engineer, Department of Public Works, and staff to the United States for the purpose of studying American sludge disposal practice. Many of the large sewage treatment plants were visited and the findings (5) of the group were as follows:

"After return of our visit to the U.S.A., we corrected our calculations of artificial sludge-drying and found a figure of more than f1.100 ($27) per ton dried sludge, for filtration and heat-drying (without digestion) amortization included.

This figure is far above the most optimistic selling price which can be expected.

The Government, who likes conservation of the organic materials, is not willing to pay the deficit. Therefore, we altered our plans of sludge disposal in accordance with your opinion and with the practice of some big places situated near the Atlantic and the Pacific coast.

As you know already we will discharge the sludge into the North Sea. We have not yet worked out this plan in details (being very busy with the more urgent and big program for the sewerage and de purification plant itself). But the chief parts of the sludge disposal plan are as follows:

The sludge will be digested in tanks situated near the coast. The (thin) digested sludge will be pumped into the sea by means of a pipe line with a preliminary fixed length of 10 kilometers (± 7 miles) from the coast line. The depth at this distance is 20 meters (± 60 ft.), the diameter of the pipe line is approximately 0.20 m. (8 in.) and the material will, most likely, be plastic (polyaethyleen).

We had a discussion with the pipe line factory in Amsterdam, who will be able to produce such a plastic line within some years. We hope to place the line on the sea bottom in a similar way as the petrol pipe line through the English Channel in 1944, coiled on a drum. (They also called "PLUTO" pipe line.)

The pumping cost is calculated on f1. 15.000 ($4,050) per annum."

This transition away from mechanization is beginning to be recognized.

Los Angeles, Calif.

At the Los Angeles Hyperion activated sludge sewage treatment plant there has been concern since initial operation with the low return from fertilizer production (digested sludge) and the high cost of mechanized sludge processing. Suhr (13) reported investigations for low cost methods of producing fertilizer, along with proposals for pumping digested sludge to sea. Engineering studies were made by several authorities which resulted in a bond issue authorizing enlargements, extensions and re-design of the system.

The basis of the new design includes a digested sludge force main 7 to 8 miles in length and a settled sewage force main about 5 miles long. The existing investment in digested sludge fertilizer equipment, costing approximately $8 million will be placed on a standby basis. There may be further testing of this equipment to determine the cost of producing activated sludge fertilizer, after plant revisions are installed.

Cost of Sludge Processing

Without considering profit, the tonnage service charge for handling digested sludge pumped to the Cleveland system from a large surburban population can be answered by computing the cost of two processes: (a) vacuum filtration, and (b) incineration.

Vacuum Filtration Costs

Rotary drum type vacuum filters with cloth media are used in Cleveland. The direct charges for operation and maintenance labor, chemicals, filter cloth, power and supplies are easily computed. These charges per ton of dry solids range from $12.26 to $13.78. The higher cost applies to low capacity units, which normally operate at a somewhat lower efficiency. Other serv-

ices furnished for vacuum filtration, such as supervision, clerical, laboratory, building and ground maintenance have been allocated from experience by the treatment plant superintendents. The costs per ton, based on the breakdown of all charges, vary from $15.25 to $17.80.

In addition to these charges, services performed by others and administrative, insurance, depreciation and interests charges have to be included. These charges, when prorated, are $11.03 to $15.30 per ton, and must be added to the treatment plant cost, making the total unit costs $26.28 to $33.10. Thus, the service charge for vacuum filtration, excluding profit, is about $30 per ton. Contemplated improvements to the process by reducing chemical costs through the use of elutriation can reduce this amount only slightly. The rising cost of labor and material quickly offset gains in process efficiency. A major reduction in the $30 cost cannot be planned for in the forseeable future. Charges are more likely to increase.

Sludge Incineration Costs

The same methods used in computing the cost of vacuum filtration have been applied to the process of sludge incineration. Multiple hearth type furnaces are used with the usual pneumatic ash conveyor system. Heat requirements, in excess of the inherent B.t.u. of the moist filter cake are furnished largely by digester gas. No charge has been made for the gas used, because the supply is usually in excess of treatment plant demands. Very little purchased fuel is used. Its cost has been included. A filtered water supply is furnished at no cost by the Division of Water and Heat and is not included in any operation charge.

Direct charges for operation and maintenance labor, power and supplies vary from $5.64 to $12.46 per ton; again the low capacity units account for the higher cost. When filter cake is given to the public, the standby charges for incineration increase the unit cost because of the reduced tonnage fed to the incinerators. Taking this into consideration, the maximum cost of direct charges would be reduced to about $6 to $7 per ton. Costs per ton based on the breakdown of all charges at the plant range from $9.74 to $14.20 for incineration.

The total unit costs, including administrative and general expense, may range from $16.79 to $26.45 per ton. Allowing for the filter cake diverted, the $26.45 charge could be reduced to $20 per ton.

This method of computation has been reasonably accurate, but still questionable from a cost accounting viewpoint. The historical cost less depreciation when compared with a valuation based on reproduction cost less depreciation has not been considered in this study. Studying financial statements from other cities has revealed that the natural tendency to minimize direct process charges will show a wide divergence when the total treatment plant costs are broken down into unit process costs.

The high tonnage costs when administrative charges are included reflect a greater allowance for annual depreciation resulting from a reevaluation study made in 1951.

There is very little accurate information on administrative charges as previously defined. Generally, treatment plant managers seem to lack communication with responsible financial officers and simply do not know to what extent administrative charges affect tonnage costs. Frequently serious error results by having services performed by others without charge to the pollution control system.

The enlarged sludge dewatering and incineration equipment to be built at Cleveland in 1957 will carry a depreciation and interest rate three times that of 1936. The common design practice of making capacity allowances for the return of digester solids to secondary treatment devices has not been eval-

TABLE II.—Vacuum Filtration Costs Per Dry Ton (1953)

Costs ($ per ton)	Washington, D. C.[1]		Columbus, Ohio[2]	
	Morgan-Thomson (14)	Schreiber (15)	Morgan-Thomson (14)	Blodgett (16)
Chemicals:				
Ferric chloride	3.10	3.10	3.79	2.96
Lime			2.54	2.18
Labor	3.91	3.91	5.32	6.50
				4.16[3]
Filter cloth	0.14	0.14	0.47	0.44
Power	0.24	0.24	0.07	1.82
Total direct charge	7.39	7.39	12.19	18.06
Other charges[4]	—	14.10	—	20.66
Total unit cost[5]	—	16.20	—	21.91

[1] Population estimate 1,040,000; sludge production 6,536 tons (dry).
[2] Population estimate 500,000; sludge production 6,630 tons (dry).
[3] Maintenance cost.
[4] Includes: supervision, laboratory, store room, machine shop, etc.
[5] Includes: administrative and general charges.

uated. No attempt was made to evaluate the cost of rehandling the digester solids in the supernatant. Solids that bypass vacuum filtration would certainly be more expensive to treat.

An answer then to the question of a service charge for ultimate disposal of digested sludge pumped to the Cleveland system is $40 to $50 per ton plus profit, based on 1953 cost records. A restriction on the sludge concentration of not less than five per cent solids would have to be imposed to avoid the high cost of recycling.

Comparison of Sludge Processing Costs

The most recent sludge processing cost information published is by Morgan and Thomson (14), who show the costs of vacuum filtration to range from $11.44 to $12.75 per ton based on a direct charge method of computation, for Buffalo, N. Y., Columbus, Ohio and San Diego, Calif., Washington, D. C., with a direct charge of $7.39 per ton had the lowest cost reported.

Schreiber (15), Superintendent of the Washington, D. C. sewage treatment plant, estimates the annual fixed charges, along with a percentage allocation to the unit processes, at $134,234 for 1947. This charge is based on a tabulation using a 65 year life for buildings and a 25 year life for equipment. Taking into consideration the 1947 valuation and applying the same unit process method of computation, a conservative total unit cost of $16.20 per ton is obtained for vacuum filtration. At Washington, D. C., about one-half the digested sludge solids reach the point of vacuum filtration. Filter cake is finally disposed of in a fill near the treatment plant site or by trucking away by interested parties at no charge.

The direct charges at Buffalo, N. Y., Columbus, Ohio, and San Diego, Calif., do not differ much from those at Cleveland. Tables II and III show the break down of charges based on cost information submitted by the respective sewage treatment plant managers. The variation in published cost information given in Table II is of particular interest.

Cost of Sludge Pumping

In order to transport concentrated digested sludge, which is practically odor free, it is necessary to acquire and

TABLE III.—Digested Sludge Incineration Costs Per Dry Ton (1953)

Costs ($ per ton)	Columbus, Ohio[1]	Cleveland, Ohio[2] (Easterly-Southerly)	Cleveland, Ohio[3] (Westerly)
Labor and material:			
Operation	3.64	3.27	9.70
Maintenance	3.51	1.73	2.14
Fuel oil	2.00	0.03	0.18
Power	1.35	0.61	0.44
Total direct charges	10.50	5.64	12.46
Other charges	12.30	9.74	14.20
Total unit cost	16.15	16.79	26.45[4]

[1] Population estimate 500,000; sludge production 4,100 tons (dry).

[2] Population estimate 1,050,000; sludge production 11,490 tons (dry).

[3] Population estimate 228,000; sludge production 1,235 tons (dry).

[4] Public used 1,569 tons of filter cake resulting in excessive standby charges for incineration. More accurate estimate would be $20 per ton.

construct the same functional elements used for coal pumping. Land easements for the pipe line and pumping stations, the cost of ditching, inspection, backfilling and restoration of easements are common construction problems. Combined operation of the pumping stations, maintenance of pipe lines and pumping stations, and overhead would considerably reduce the cost of transportation of both commodities. Another approach to this problem is that the combined operation cost would be the same as that of a single coal line of much greater pumping capacity. Greater tonnage transported by pipe line sharply reduces the operation costs. The problem of cost depends largely on the tonnage pumped. For long distance pumping (50 to 100 miles) a connected population of perhaps two million people would be an economical starting point.

Pumping Costs at Cleveland

The cost of pumping raw sludge with more than 3 per cent dry solids a distance of 13 miles has averaged $1.32 per ton during the period 1940 to 1954. The breakdown of the total cost is as follows:

Item	Cost ($ per dry ton)
Power	0.129
Operation Labor	0.168
Maintenance*	0.200
Experimental	0.073
Depreciation (@ 2.0%)	0.375
Interest (@ 2.0%)	0.375
Total	1.320

* Includes capital, equipment, and replacement costs at the pumping station.

The pumping of concentrated raw sludge is complicated by the separation of fats and grease in the force main. Cleaning has been necessary 8 to 10 times per year in order to remove the deposited grease, scale and silt. Since the pumps are subject to abrasive conditions, their life is relatively short. Replacement of equipment at the pumping station has been carried under maintenance and not capitalized. A total cost of $0.10 per ton mile, including interest and depreciation is obtained for raw sludge when pumping 30,000 tons per year. Additional

TABLE IV.—Raw Sludge Pumped

Year	Total Volume (m.g.)	Settled Sewage (gal./m.g.)	Dry Solids (%)	Dry Solids (tons)	Dry Solids ($/ton)
1940	252.50	8,400	2.97	31,205	0.45
1941	219.59	8,450	3.04	28,760	0.55
1942	297.73	9,380	2.51	30,391	0.45
1943	298.76	8,690	2.41	28,588	0.58
1944	220.43	6,375	3.30	30,094	0.63
1945	224.16	6,814	3.30	29,803	0.50
1946	214.64	6,473	3.50	30,437	0.40
1947	213.93	5,888	3.80	32,684	0.48
1948	226.48	6,194	3.30	31,598	0.43
1949	215.38	5,919	3.40	30,368	0.52
1950	214.26	5,899	3.10	27,980	0.64
1951	190.99	5,302	3.30	26,627	0.87
1952	179.39	5,340	3.40	24,894	0.65
1953	206.88	6,026	3.10	26,594	0.71
1954	203.93	5,690	3.50	30,071	0.67
Total	3,379.05	100,840	47.93	440,094	8.53
Av.	225.27	6,723	3.19	29,340	0.57
Max.	298.76	9,380	3.80	32,684	0.87
Min.	179.39	5,302	2.41	24,894	0.40

TABLE V.—Total Expenditures For Pumps and Force Main

Year	Operation	Maintenance	Power	Total
1940	$3,762.31	$ 6,160.77	$4,225.12	$14,148.70
1941	3,946.27	7,775.61	3,965.83	15,687.71
1942	4,294.49	5,562.39	3,901.84	13,758.72
1943	4,528.36	7,725.37	4,237.89	16,491.62
1944	4,638.90	10,806.85	3,554.35	19,000.10
1945	4,552.30	7,549.12	2,652.87	14,754.29
1946	3,844.95	5,554.49	2,626.86	12,026.30
1947	4,322.27	8,531.32	2,849.28	15,702.87
1948	4,626.51	5,485.62	3,350.88	13,463.01
1949	4,780.98	7,806.02	3,340.60	15,927.60
1950	5,592.19	8,559.08	3,697.02	17,848.29
1951	5,681.14	13,184.85	4,319.77	23,185.76
1952	6,251.90	6,059.24	3,978.74	16,289.88
1953	5,933.00	8,208.34	4,833.61	18,974.95
1954	6,125.55	9,420.40	4,636.80	20,182.75
Total	$72,881.12	$118,389.47	$56,171.46	$247,442.55
Av.	4,858.74	7,892.63	3,744.76	16,496.17
Max.	6,251.90	13,184.85	4,833.61	23,185.76
Min.	3,762.31	5,485.62	2,626.86	12,026.30

tonnage added to the force main can be transported at approximately $0.02 per ton mile. The ultimate capacity of this particular line using mixed raw and digested sludge is approximately 80,000 tons per year.

Tables IV and V give the annual operating expenditures and quantities for the years 1940 through 1954. Depreciation and interest charges were calculated at 2 per cent each on a capital charge of $568,000. Total depreciation plus interest per ton is $0.75. The costs of testing and development included under experimental have been good investments. Generally, unit process costs have more than doubled over the same period while the cost for sludge pumping has remained almost constant.

Disposal of Garbage

The addition of ground garbage to sludge digestion tanks is frequently provided for in the design of treatment works. As a result, the concentration of solids in the digestion process is increased in proportion to the solids added. The total tonnage of digested solids may increase 50 to 100 per cent. The addition of garbage favorably influences both the solids concentration and tonnage in force main pumping.

The present cost of garbage disposal by incineration is of the same order as sludge solids. In addition, many metropolitan centers are frequently unable to obtain land for new incinerator construction. Problems of allied air pollution must also be considered.

Proposed Force Main System

The success of any mass movement of commodities depends upon the needs or demands of at least two widely separated and well established locations. The case of mass movement of coal is quite clear; an almost unlimited supply has been developed at the strip mine area and an established major consumer at the power station, 108 miles distant. Connection of the two by a force main is clearly the result of sufficient technical information to support the economic aspects of coal pumping. In the case of digested sludge pumping, the needs of the congested area surrounding Cleveland, Ohio are recognized as urgent. Consulting engineers are bewildered with the burdens of drainage area problems and handcuffed with municipal boundary lines. Current design practice is inadequate to meet the needs of an expanding economy. Whether we like it or not, inter-city or regional planning is the only answer. Cleveland with its satellite cities and surrounding industries is admirably situated to form the northern anchor of a 100 mile digested sludge force main.

To the south, at a distance of nearly 80 miles, the strip mine area begins. The estimated capacity of a single strip mine trench is equal volumetrically to all the lagoons of the Chicago Sanitary District. Chicago has lagooned 567,662 tons of dry solids in 7 years, or about 14 lb. per cubic foot.

The abandoned mine areas are costly to maintain because of taxes and legislative requirements covering reforestation. Acid bearing runoff from strip mines is a continuing source of pollution that could be partially stopped by the addition of highly alkaline digested sludge. The sealing of exposed sulfur

bearing surfaces in the trenches with digested sludge would prevent further oxidation. These huge wasted storage spaces could become a valuable asset rather than a liability to the mine owner.

Restoration of fertility to thousands of acres would indeed be a popular issue. The slow consolidation of digested sludge in deep lagoons would provide in 15 to 20 years a future source of beneficial organic material. Some regulations may be necessary for the protection of ground water supply. The storage of digested sludge should be a planned program and haphazard pumping should be avoided.

Figure 1 shows a proposed layout for the dual coal-digested sludge pipe line system. All of the sewage treatment works utilize separate sludge digestion. Some, such as Canton, Ohio, have pro-

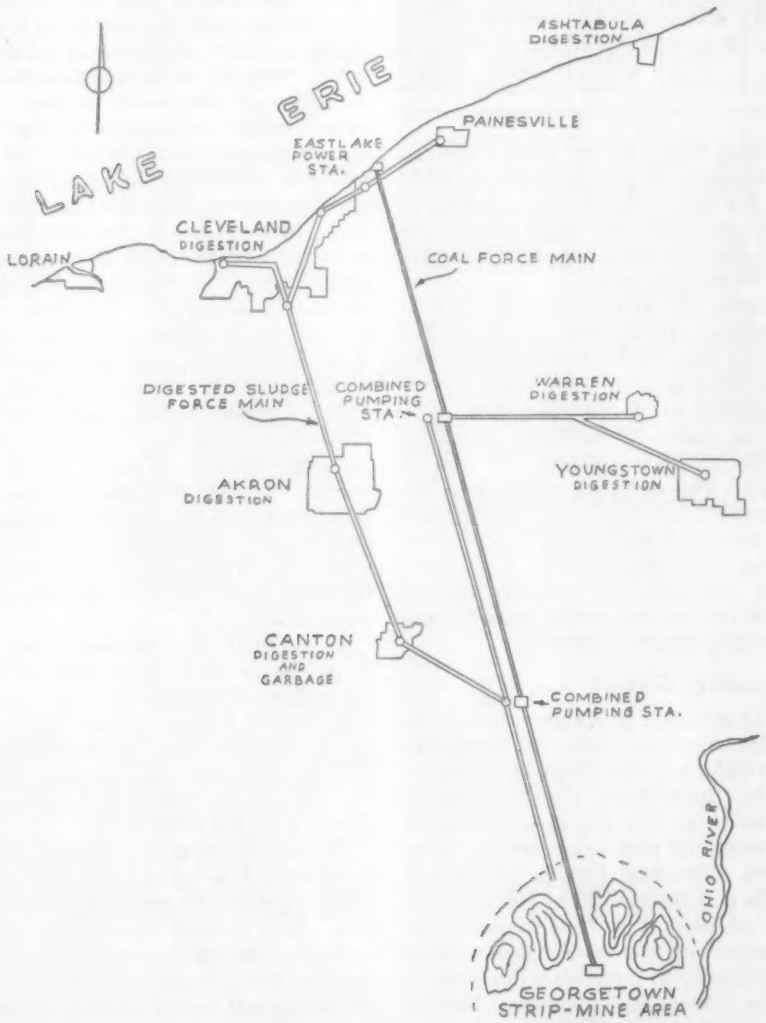

FIGURE 1.—Sketch of a proposed coal-digested sludge pipe line system.

vided for the digestion of ground garbage. In this plan, it is estimated that the service charge for pumping digested sludge into the system would be \$0.10 to \$0.15 per ton mile. Cleveland would be the major collection point and has the advantage of a 13-mile force main with adequate capacity for future demands. With this advantage, Cleveland could accept digested sludge from the satellite cities at considerably less than the \$40 to \$50 per ton cost shown to be necessary under present conditions.

Conclusions

Based on the information available, it appears that digested sludge, especially in large tonnages, can be transported economically in pipe lines to remote areas for ultimate disposal.

A private corporation, experienced in the field of mass transportation of commodities by pipe line, is recommended for this intercity service. The acquisition of sufficient annual tonnage from several cities could be most effectively accomplished by a private corporation. The Ohio Pipe Line Act, 1723.01, Revised Code, would provide the corporation the necessary law.

For ultimate disposal of ground garbage, combustible wastes and compatible industrial wastes, more actual pumping performance data is needed before these wastes could be included in the proposed mass transportation system.

Although interprocess pumping experience dates from about 1900, the economical side of the matter has never been explored thoroughly. It is for this reason and the fact that the field of application presents economic problems foreign to cities that limited use has been made of the process.

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COMPARATIVE ANALYSES OF SEWAGE SLUDGES

By M. S. Anderson

Senior Chemist, Soil and Water Conservation Research Branch, Agricultural Research Service, United States Department of Agriculture, Beltsville, Md.

Modern developments in sewage treatment plant construction and operation make desirable another look at the products that are potentially available for use as fertilizers and soil conditioners in agriculture.

It is well known that biochemical reactions take place at a rapid rate in sewage sludge. The microorganisms involved are active from the time material enters a sewer until dried sludge is prepared. The end products of sewage treatment have often been shown to vary widely in chemical composition and in agricultural usefulness, depending on the processing operation. Sludge from the activated sludge process has sufficient nutrient value to serve as a fertilizer or fertilizer material. Sludge from anaerobic digestion, regardless of possible previous treatment, has a relatively low level of available plant nutrients. These sludges are sometimes used, however, as constituents of mixed fertilizers and for direct application to the soil.

It is the purpose of this paper to examine some sewage solids and sludges obtained at different stages in the treatment of sewage. In this way it is possible to determine something of the character and rate of change taking place, as well as the influence of different chemical components of the raw sludge on the composition of the end product.

During recent years several developments have tended to change the chemical composition of the incoming sewage. Among the factors involved are: (a) various industrial wastes; (b) the widespread use of synthetic detergents with a highly soluble phosphate content; and (c) the increase in the use of home garbage grinders. Home garbage grinders reduce bones to sufficiently small particle size so that these highly phosphatic materials are processed with the sewage.

Two basic types of sludges were studied (1): (a) dried undigested sludge from the activated sludge process and (b) dried digested sludge from both primary treatment and activated sludge facilities.

Phosphorus

Table I shows the large increase during the past 20 years in the phosphorus content of sludges from the activated sludge process as revealed by the phosphoric oxide present in the sludges. The data in Table I for digested sludges show some increase in phosphorus content but the increases are not as large proportionately as are shown for activated sludge. A period of about 20 years has brought about

TABLE I.—Comparison of the Nitrogen and Phosphorus Content of Activated Sludge and Digested Sludge, 1931–35 and 1951–55

Determination	Activated Sludge		Digested Sludge	
	1931–35	1951–55	1931–35	1951–55
Nitrogen (%):				
Min.	4.4	4.8	1.3	1.8
Av.	6.0	5.6	2.2	2.4
Max.	6.4	6.0	3.0	3.1
Phosphorus [P_2O_5] (%):				
Min.	2.0	4.0	0.8	0.9
Av.	3.2	5.7	2.1	2.7
Max.	3.8	7.4	3.8	5.0

TABLE II.—Minor Chemical Elements in Activated Sludge and Digested Sludge

Sludge	Element				
	Copper	Zinc	Boron	Manganese	Molybdenum
Activated sludge (p.p.m.)					
Min.	385	950	6	65	6
Av.	916	2,500	33	134	16
Max.	1,500	3,650	74	190	45
Digested sludge (p.p.m.)					
Min.	315	1,350	4	30	2
Av.	643	2,459	9	262	6
Max.	1,980	3,700	15	790	12

little change in the average nitrogen contents of sludges from either activated sludge treatment or anaerobic digestion processes.

In order to characterize present day sewage sludges from a chemical standpoint, summarized values for minor elements contained in activated and digested sludges are given in Table II. The elements copper, zinc, boron, manganese and molybdenum each show wide variations in the different sludges. These differences probably reflect the effect of industrial wastes entering the system as well as various materials contributed with the domestic sewage.

Nitrogen

The availability of nitrogen to plants when sludge is placed in the soil, is an item of great interest to prospective users of sewage sludge as a fertilizer. Determinations of rates of nitrification in sludge are made usually after specified periods of incubation with soil. These data have been obtained from sludges representative of different treatment systems. Average values are shown graphically in Figure 1. The data confirm the fact that the nitrogen in the undigested sludge from the activated sludge process is more readily nitrified than is the nitrogen in digested sludge (2) (3). These data also show that most of the nitrification accomplished during a growing season takes place within a month after the sludge is incorporated with the soil. The graphs for the two sludges are approximately the same in character but represent different degrees of nitrification.

Activated sludge has a total nitrogen content (Table I) about two and one-half times the amount in digested sludge and a rate of nitrification also about two and one-half times that of digested sludge. If the fertilizer value of sewage sludge was based entirely on the content of quickly nitrifiable nitrogen, the value of undigested sludge from the activated sludge process, based on weight, should be about six times that of digested sludge. Any relative evaluation, however, must take into account the character of the soil to which the fertilizer is to be added and crop to be grown.

Chemical data on the influent solids and the sludges from each of the sewage treatment plants studied are given

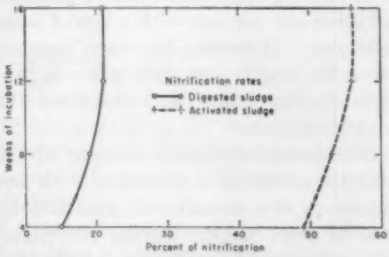

FIGURE 1.—Rates of nitrification of digested sludge and activated sludge.

TABLE III.—Chemical Composition of Sewage Sludges Studied. Dry Weight Basis

Sewage Treatment Plant	Nitrogen (%)	Carbon (%)	Carbon-Nitrogen Ratio	Phosphoric Oxide (%)	Ash %	pH
Washington, D. C.[1]						
Influent solids (spring)	2.42	43.46	18.0	1.14	32.35	5.3
Influent solids (summer)	2.39	43.69	18.3	1.09	37.59	5.6
Digested sludge	2.06	28.59	13.9	1.44	52.83	5.8
Baltimore, Md.						
Influent solids	2.23	47.09	21.1	1.29	24.16	5.7
Activated sludge	2.36	30.37	12.9	11.01	29.70	5.5
Humus tank sludge	5.34	37.90	7.1	3.96	32.30	5.9
Heat-dried dig. sl.	3.05	36.53	12.0	2.97	39.73	5.0
Jasper, Ind.[2]						
Influent solids	2.90	42.31	14.6	1.62	32.29	5.7
Activated sludge	3.51	23.01	6.6	2.81	52.43	6.8
Digested sludge	5.89	22.95	3.9	3.49	36.96	5.8
Richmond, Ind.[3]						
Influent solids	3.80	28.21	7.4	5.19	40.94	7.6
Activated sludge	3.02	44.04	14.6	3.64	31.37	6.2
Digested sludge	2.24	26.36	11.8	4.34	50.09	6.9
Chicago, Ill. (Southwest plant)[4]						
Raw sludge	2.70	46.62	17.3	2.71	28.24	4.5
Activated sludge	4.98	28.62	5.7	5.58	34.82	6.2
Heat-dried sludge	5.56	29.41	5.3	6.56	37.42	6.0
Milwaukee, Wis.[4]						
Heat-dried sludge	5.96	20.88	3.5	3.96	27.73	4.8

[1] Primary treatment.
[2] Prevalence of home garbage grinders affects sludge composition (6).
[3] Ground garbage discharged directly to digesters.
[4] Sludge not digested.

in Table III. The data show several features worthy of attention. Aeration causes an increase in the nitrogen content of the sewage solids. These relatively high values are presumably built up in part by bacterial action transforming water-soluble nitrogen into the bodies of the organisms. It is reported that these organisms contain 7 to 11 per cent nitrogen (4).

Anaerobic digestion tends to produce a higher ash content with a loss of some nitrogen. However, digestion does reduce the number of pathogenic organisms in the sludge and stabilizes the organic material.

The phosphoric oxide content of the sludges varies in accordance with the intake of this constituent, modified by loss in total weight during the digestion process. A small amount of phosphorus may be expected in the effluent.

Carbon-Nitrogen Ratio

Carbon-nitrogen ratio values that are so widely used by soil scientists serve as an approximate measure of the extent of decomposition of organic matter taking place during the different stages of the treatment process. The high carbon-nitrogen values for raw sewage (Table III) reflect the presence of large amounts of carbohydrates. The rapid decrease in carbon content is indicative of decomposition. Time factors are not definite, but aeration probably changes carbon relations more rapidly than does the anaerobic digestion process. The former produces a lower carbon-nitrogen ratio than is accomplished by digestion.

Organic carbon was determined by the Walkley-Black wet combustion method (5). Calculations are based on the assumption that 77 per cent of the

carbon present is recovered. This value, developed for soils, may be subject to greater error when applied to sludge materials. Consequently the carbon values are probably a little higher than would be the case if dry combustion data were used.

pH

Sludge pH values are influenced to some extent by the addition of sludge conditioning materials such as ferric chloride. Variations in sludge processing and the composition of the incoming solids may also affect the pH. The pH value of the sludge may in some cases influence its use where the application of neutral or alkaline materials to the soil must be avoided.

Conclusions

It is apparent that additional study is needed to understand better the value of sewage sludge for agricultural use. The following topics are indicative of the need for research on the subject.

1. To what extent is the phosphorus in sewage sludges available to plant life?
2. Do the specific feeding powers of various plants differ with respect to their utilization of phosphorus in sewage sludges?
3. What chemical characteristics of sludges most effectively influence the rate of nitrification?
4. Biology of the trickling filter.
5. How is the high nitrogen content developed in activated sludges?

Acknowledgment

Sincere appreciation is expressed for the wholehearted cooperation and assistance by representatives of the various sewage treatment plants who furnished samples and the staff members of the Soil and Water Conservation Research Branch and the Soil Conservation Service of this Department who made analyses.

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MARCH DIRECTORY

A supersize volume is scheduled for the March issue of THIS JOURNAL. The annual directory number reflects the admirable growth of the Federation Member Associations since the last complete directory was issued in March, 1954. Approximately 6,800 listings will feature complete names, addresses and professional occupations as of December 31, 1955.

In addition to the complete listing of members and the Annual Yearbook, the 1956 Directory will include for the first time a Geographic Listing of the membership by states and cities.

THE RELATIVE STABILITY TEST

By William L. Tidwell* and J. H. Sorrels

Assistant Research Bacteriologist and Research Engineer, Respectively, Texas Engineering Experiment Station, Texas A. & M. College System, College Station, Texas

One of the oldest tests used today for the analysis of sewage is the relative stability test. It was first introduced to this country by Phelps and Winslow in 1907 (1) and was modified by Phelps in 1908 (2) to give quantitative results. The result obtained in this test is defined as the per cent ratio of oxygen available (as dissolved oxygen, nitrite and nitrate oxygen) to the total oxygen required to satisfy the biochemical oxygen demand.

During the past years, this test has been used less and less, a state which may be considered not necessarily desirable. It is believed that the test still has a distinct place in the series of tests run to determine the strength of sewage effluents. It should be used as an adjunct to the more exacting tests, such as the B.O.D. test, or alone, if facilities for the more exacting tests are not available.

Probably one of the reasons for the lesser use of this test is the lack of understanding of its method of action. Some of the books describing the method of this test either do not give any explanation of how it works, or cite a now-known incorrect theory of its method of action. This old theory, still widely repeated, states that the loss of color of the methylene blue is due to the production of sulfur compounds, such as hydrogen sulfide, under the anaerobic conditions present when the available oxygen has been exhausted. The incorrectness of this theory and the actual method of action of this test were presented by Wooldridge (3) and Wooldridge and Standfast (4). Unfortunately, their papers were published in a journal not readily accessible to the sanitary engineer and seem to have been largely overlooked. A review of this method of action and a laboratory demonstration of factors affecting the relative stability of sewage are the purposes of this paper.

Method of Action

Wooldridge (3) and Wooldridge and Standfast (4), in a series of well-executed experiments, were able to demonstrate that the results of the relative stability test were due to the action of a group of oxidizing enzymes, called dehydrogenases. The way in which these enzymes function can be stated simply. Instead of oxidizing a substance by adding oxygen to it, they oxidize by removing hydrogen from the substance. This removal of hydrogen results in oxidation, since the substance now has less hydrogen (and thus proportionally more oxygen) than it had before. The hydrogen removed is then transferred to some other compound which is thereby reduced. There are many compounds capable of accepting this hydrogen. Among them are oxygen, nitrites, nitrates and methylene blue. In this reaction, the oxygen is converted to water, the nitrites and nitrates to nitrogen or ammonia, and the methylene blue is converted to leuco-methylene blue, which is colorless. These enzymes are produced by the bacteria, providing their principle method of obtaining energy.

In the relative stability test bottle are all the components necessary for the operation of the dehydrogenase sys-

* Present address: Department of Biological Sciences, San Jose College, San Jose, Calif.

tem. The organic matter of the sewage serves as the source of material to be oxidized; the bacteria furnish the enzymes; and the dissolved oxygen, nitrates, and nitrites serve as hydrogen acceptors. When the "available oxygen" is exhausted, the methylene blue accepts the hydrogen and becomes decolorized.

A critical study needs to be made on exactly what constitutes the "available oxygen" mentioned above, since there is some disagreement on the timing as to when the various hydrogen acceptors begin to be reduced. Some researchers interpret the process as being step-wise while others suggest that all the reductions are occurring simultaneously but probably at different rates.

Factors Affecting Relative Stability

From the foregoing discussion, it is obvious that there are a number of factors that could affect the relative stability reading since this is an enzyme reaction involving a series of components. Some of these factors would be the temperature, the concentration of methylene blue, the quantity of dissolved oxygen, the B.O.D. of the sewage, the concentration of nitrates and nitrites and the amount of enzyme or the number of bacteria present. The first two factors are, of course, controlled by the standard method established for running this test. The other factors, however, are uncontrolled and it is their integrated effect that determines the relative stability of a sewage.

In order to demonstrate the effect of varying these normally uncontrolled factors, a series of tests were carried out in which all but one of these factors were kept constant and the effect of the varied factor on relative stability was noted. These tests were carried out using the following preparations:

1. Synthetic sewage. In order to have a source of organic material, constant in its concentration, with no solids to cause variability, the artificial sewage described by Butterfield *et al.* (5) was used.

2. Bacterial suspension. For the sake of uniformity again, a special bacterial suspension was prepared for use as an inoculum. A small sample of raw sewage was collected and passed through a fine-pore, sintered glass filter to remove any protozoa present. One milliliter of this filtrate was then used to inoculate three flasks of nutrient broth, each containing 100 ml. The flasks were incubated for 24 hr. at 37° C. and the bacterial growth concentrated by centrifuging for one hour at 2,500 r.p.m. The broth was then decanted from the centrifuge tubes and the bacteria suspended in normal saline to wash any remaining broth from them. The bacteria were again concentrated by centrifugation as before and the saline wash decanted. The bacteria were once more suspended in saline, large clumps allowed to settle, and used as the inoculum.

3. Aerated dilution water was used as a diluent to obtain the necessary concentrations of synthetic sewage and dissolved oxygen to simulate sewage treatment plant effluents.

4. Standard methylene blue solution.

5. A stock solution of sodium nitrate containing one milligram per milliliter.

In order to demonstrate the effect of a varying concentration of organic matter on the relative stability of sewage, a series of bottles was filled with

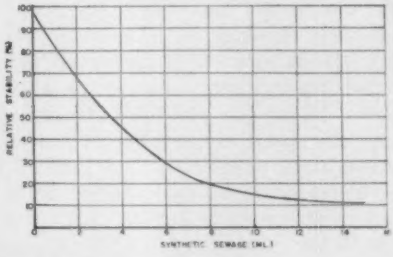

FIGURE 1.—Effect of organic matter on relative stability.

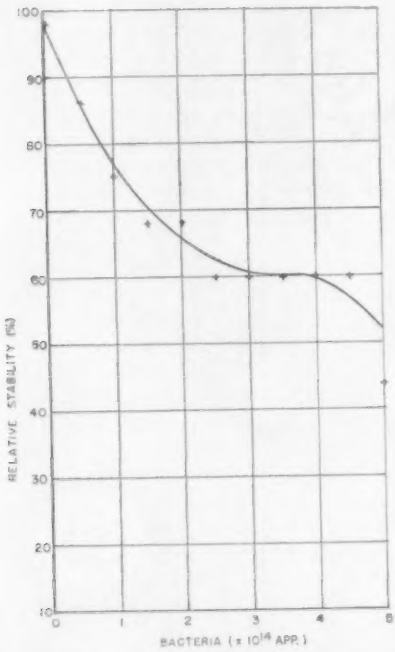

FIGURE 2.—Effect of number of bacteria on relative stability.

fixed amounts of methylene blue, inoculum, and dilution water while the concentration of synthetic sewage was varied from bottle to bottle. All bottles were incubated at 20° C. until the methylene blue was decolorized. Results, shown in Figure 1, demonstrate clearly the decrease in relative stability with an increase in the concentration of organic matter. In a similar fashion, holding all factors constant except the numbers of bacteria, it was shown (Figure 2) that an increase in the amount of enzyme (number of bacteria) present would also cause a decrease in relative stability. By varying only the concentration of the nitrates, relative stability was shown to be increased by an increase in nitrate concentration (Figure 3). That this occurs in the presence of either a high or a low concentration of organic matter is also shown.

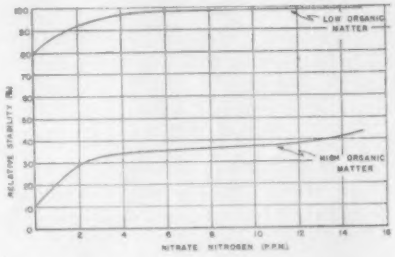

FIGURE 3.—Effect of nitrate nitrogen on relative stability at two levels of organic matter.

Attempts to demonstrate the effect of dissolved oxygen on relative stability using the previously mentioned method were unsuccessful. Several attempts, using varying combinations of aerated and unaerated water, met with failure. Since dissolved oxygen and relative stability could not be run on the same sample, each combination had to be set up in duplicate and exact measurements were virtually impossible due to minor variations in the bottle sizes. Moreover, even when this was accomplished approximately, the results did not produce good curves because as the

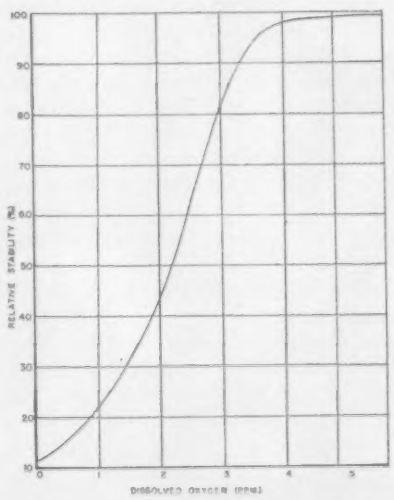

FIGURE 4.—Relationship between relative stability and dissolved oxygen.

concentration of dissolved oxygen was increased, the numbers of bacteria also increased during the incubation period. Since the two had opposite effects on the relative stability, the results had little significance. This effect can be seen, however, in Figure 4 which is based on actual sewage treatment plant data.

Discussion

As it has been previously stated and partially shown, the integrated effect of the amount of organic matter, the number of bacteria, and the concentrations of dissolved oxygen, nitrates and nitrites determines the relative stability of a sewage. It is possible, therefore, for one of the factors to be high without being reflected in the relative stability if the other factors are such as to offset it. For example, one normally expects a sewage high in B.O.D. to have a low relative stability. However, if the sewage had a high concentration of dissolved oxygen and nitrates and a low bacterial count, the relative stability might also remain high. Conversely, a sewage low in B.O.D. is normally expected to have a high relative stability, but if this same sample is low in oxygen and nitrates, with a high bacterial count, the relative stability probably would be low.

Summary

1. The method of action of the relative stability test as shown by Wooldridge (3) and Wooldridge and Standfast (4) is discussed.

2. Factors affecting the relative stability reading are discussed and their effects demonstrated experimentally. The fact is revealed that relative stability varies indirectly with the amount of organic matter and the number of bacteria and directly with the concentration of nitrates. By using sewage treatment plant data, the effect of dissolved oxygen on relative stability is demonstrated.

Acknowledgment

The technical assistance of Mrs. Barbara J. Tongate in these studies is gratefully acknowledged.

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PERFORMANCE OF A NEW TYPE MECHANICAL AERATOR

By K. L. Schulze

Civil Engineering Department, Michigan State University, East Lansing, Mich.

The first two-stage sewage treatment plant using a mechanical aeration system * developed by the author started operation on April 1, 1954 at New Holstein, Wis. A description of the treatment plant and a report on the initial operation results achieved during 1954 have been published elsewhere (1). A schematic diagram of the installation is shown in Figure 1. Four aeration tanks were constructed, although only two units are used during normal operation when the canneries are not in production.

Two of the four aeration tanks are normally operated in series. The B.O.D. load from the municipal waste varied from 200 to 400 lb. per day and was handled by two aerators which delivered air at the rate of 9 and 11 c.f.m., respectively. The amount of air delivered by a belt driven aerator can be measured conveniently in the field by installing a rotary seal on top of the hollow shaft and connecting the seal with an air metering instrument. During the pea canning season, when the B.O.D. load increased to approximately 2,000 lb. per day, all four aeration tanks were operated and units No. 3 and 4 were sped up to deliver air at 20 c.f.m. each. This is the same air volume that units No. 1 and 2 were delivering together. It seemed reasonable, after the canning season had ended, to try to handle the municipal waste with only one of these newer aerators.

The experiment started on January 25, 1955 using aeration tank No. 4. The tank capacity is 2,720 cu. ft. or 20,000 gal., and the tank dimensions are 16.5 ft. × 16.5 ft. × 10 ft. The aerator is driven by a 7.5 hp. motor at 700 r.p.m. and delivers 20 c.f.m. of air. Daily 24-hr. composite and proportionate samples on both the primary and the final effluent were collected and analyzed. The results of these tests are given in Table I and have the following average values:

Flow (g.p.d.)	136,000
B.O.D. (p.p.m.):	
Primary effl.	186
Final effl.	16.5
Reduction (%)	91.1
Load, aeration system (lb./day)	212
Air supply (cu. ft./lb. B.O.D./day)	136
Aeration tank capacity (cu. ft./lb. B.O.D.)	12.8
Aeration time, 24-hr. flow (hr.)	3.5

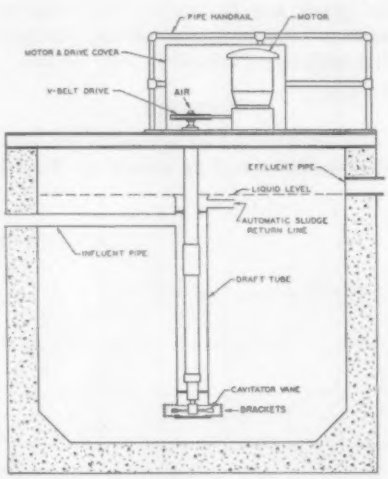

FIGURE 1.—Schematic diagram of the aeration unit (Cavitator).

* "Cavitator" unit, manufactured by Yeomans Brothers Company, Melrose Park, Ill.

TABLE I.—Test Results from a Single-Stage Aerator Treating Municipal Wastes [1]

Date 1955	Flow (1,000 g.p.d.)	B.O.D.					Aeration		Air *versus* B.O.D.		Kw hr.[3]	D.O. Mixed Liquor (p.p.m.)	Susp. Solids (p.p.m.)	Sludge Index (ml./g.)	Settl. Solids (ml./l.)
		Prim. Effl. (p.p.m.)	Final Effl. (p.p.m.)	Load (lb./day)	Removal		Tank Cap.[2] (cu. ft.)	Time (hr.)	cu. ft./lb. B.O.D. Load per day	cu. ft./lb. B.O.D. Removed per day					
					(lb./day)	(%)									
Feb.															
10	113	264	36	249	215	86.4	11	4.25	115	134	—	2.9	—	—	575
11	97	162	18	131	116.5	88.9	20.8	4.95	220	247	—	2.3	—	—	600
15	107	222	6	198	192.5	97.3	13.7	4.5	145	150	—	1.9	—	—	500
16	105	156	9	136	128	94.2	20	4.57	212	225	—	1.9	—	—	580
17	104	130	18	113	97.5	86.2	24	4.6	254	295	—	2.5	—	—	570
18	107	186	15	166	152.5	91.9	16.4	4.5	173	189	—	2.7	—	—	585
24	122	186	18	189	170.5	90.3	14.4	3.9	152	169	—	9.8	—	—	40
25	124	156	27	153	126.8	82.7	17.7	3.8	188	226	—	3.0	—	—	285
28	122	194	3	197	194	98.5	13.8	3.9	146	148	—	2.0	3,476	128	445
Mar.															
1	145	228	9	276	265	96	9.8	3.3	104	109	0.51	2.3	3,640	120	435
2	145	132	12	160	145	90.9	17	3.3	180	200	—	1.9	4,670	97	455
4	193	180	18	290	261	90	9.4	2.5	99	110	0.52	3.5	3,040	118	360
7	143	216	27	258	226	87.5	10.5	3.4	111	127	0.60	5.4	3,168	85	270
8	151	198	12	250	235	94	10.9	3.2	115	122	0.57	4.0	3,448	101	350
9	150	186	15	233	214	92	11.7	3.2	123	135	0.63	4.0	2,954	127	375
10	244	192	21	390	347	89.1	7.0	1.9	74	83	0.39	5.3	2,950	85	250
Av.	136	186	16.5	212	193	91.1	12.8	3.5	136	149	0.537	3.48	3,420	108	415

[1] Sludge return rate: 37 per cent.
[2] Cubic feet per pound of B.O.D. per day.
[3] Kilowatt hours per pound of B.O.D. removed per day.

The design data specified an air requirement of 135 cu. ft. per pound of B.O.D. removed per day and an aeration period of 4 hr., based on raw waste flow, or 15 cu. ft. of aeration tank capacity per pound of B.O.D. load per day.

The test results indicated that the design data which had been tentatively set for this type aeration unit in the treatment of domestic waste were fully justified. In addition, the data accumulated provide more detailed information on the applied sludge return rate, power requirement, the amount of suspended solids carried in the mixed liquor and the biological loading factor.

At the New Holstein sewage treatment plant the return sludge was pumped intermittently at a rate of 230 g.p.m. to the influent wet well where it was mixed with the primary effluent and then fed to the aeration unit. The sludge pump was operated by an adjustable electrical time control which was set to pump 4.5 min. every 30 min. The sludge return, therefore, amounted to 50,700 g.p.d. which is 37 per cent of the average waste flow. The total flow through the aeration tank was 186,700 g.p.d. which reduced the actual aeration time from 3.5 to 2.56 hr. The final settling tank contains 27,900 gal. which provided 4.9 hr. detention time based on the raw waste flow, and 3.6 hr. based on the actual flow through the unit. The reason for these comparatively long detention times is that the settling tank had to be designed for the higher flows during the pea canning season.

Power requirement naturally depends on the B.O.D. load handled by the treatment plant. To get an average power demand near the actual capacity of the system, those days were selected when the treatment plant removed more than 200 lb. of B.O.D. per day at an efficiency of 90 per cent or better. The data in Table I show that 0.537 Kwhr. were used to remove 1 lb. of B.O.D. per day. This is slightly higher than the average of 0.4 Kwhr. per lb. of B.O.D. removed per day for standard aeration (2). Part of this disadvantage is offset by the smaller aeration tanks that may be used with this type aeration unit. Furthermore, there are indications that it will be possible to reduce the power requirement to a considerable extent by future developments.

Oxygen Utilization

At the present time an increasing number of investigators recognize that the amount of oxygen supplied to the mixed liquor is the most important limiting factor in biological waste treatment. It is of interest, therefore, to know the minimum amount of air necessary for the removal of 1 lb. of B.O.D. per day under a given set of conditions. The average value for this air requirement is 149 cu. ft. with a minimum of approximately 110 cu. ft. This minimum value is in good agreement with the data obtained in May to July, 1954, when the treatment plant was treating municipal and cannery waste using two and four aeration tanks (1). Plotting per cent B.O.D. removal versus air supply per pound B.O.D. load per day resulted in a curve showing that 110 to 130 cu. ft. of air were sufficient to obtain 90 to 94 per cent B.O.D. removal under these varied conditions. Assuming this value is valid, then a convenient method is available for determining the maximum amount of B.O.D. that can be removed by a given aeration unit. In this case, with an air delivery of 28,800 cu. ft. per day, the maximum B.O.D. removal should be 262 lb. per day. The maximum load on the aeration system at 90 per cent B.O.D. removal would be 290 lb. of B.O.D. per day. The results (Table I) on March 4 and 10, 1955 indicate that the treatment plant was actually able to handle satisfactorily a B.O.D. load of 290 and 390 lb., respectively. In order to substantiate the loading limit,

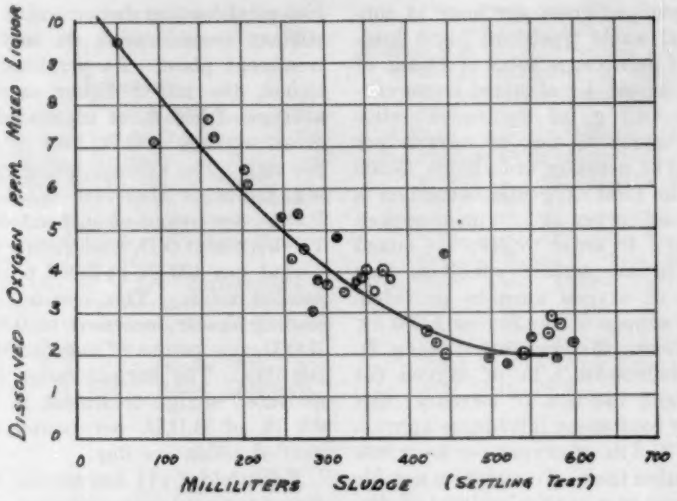

FIGURE 2.—Relationship between sludge concentration in the mixed liquor (as measured by the settling test) and the dissolved oxygen.

it would be necessary to operate the plant at a B.O.D. load of 300 to 400 lb. per day for a considerable period of time. Furthermore, the minimum air requirement of 110 cu. ft. per pound of B.O.D. removed per day mentioned previously is intended to be valid only for domestic waste. Tests with dairy and meat packing waste have shown that in these cases higher amounts of air are required per pound of B.O.D. removed (3) (4).

Generally it can be concluded that the aeration system, even using only one aeration unit, is able to handle more than an average B.O.D. load of 212 lb. per day. Table I shows that during the test period the dissolved oxygen content in the mixed liquor never decreased below 1.9 p.p.m., the average being 3.48 p.p.m. Even at 600 ml. of sludge per liter of mixed liquor, aerobic conditions were maintained. Figure 2 shows that a definite relationship exists between the amount of suspended solids carried in the mixed liquor and the dissolved oxygen. The milliliters of sludge per liter of mixed liquor, as recorded by the settling test, are plotted against the dissolved oxygen of the supernatant as measured by the Winkler method. The tests were made simultaneously on the same sample. It would be better to plot the dissolved oxygen against the suspended solids concentration rather than the settling test. But due to lack of equipment, data on the suspended solids concentration were available only for the second part of the experiment. These data (Table I) show that the mixed liquor carried between 2,950 and 4,670 p.p.m. suspended solids with an average of 3,420 p.p.m. The sludge index averaged 108 ml. per gram, indicating that the floc was of normal density and settleability.

A total of 556 lb. of activated sludge in the aeration tank is indicated by the presence of 3,420 p.p.m. of suspended solids. The oxygen utilization rate of normal activated sludge has been determined by Bloodgood (5) to range from 14 to 20 mg. of oxygen per gram per hour. This is in agreement with more recent data given by Wuhrmann (2) who found the oxygen uptake rate to range from 11 to 20 mg.

of oxygen per gram per hour at conventional waste treatment plant loadings. If an average value of 14 mg. of oxygen is used, 1 l. of mixed liquor containing 3.42 g. of suspended solids utilizes about 48 mg. of oxygen per hour. The aeration tank holds 75,500 l. and the total oxygen consumption is 3.62 kg. of oxygen or 7.96 lb. of oxygen per hour. In order to keep the mixed liquor in an aerobic condition this amount of oxygen must be provided. The air supply is 20 c.f.m. or 1,200 cu. ft. per hour. Assuming that 55 cu. ft. of air represents 1 lb. of oxygen (at 32° F. and 740 mm. of mercury) this aeration equipment introduces approximately 21.8 lb. of oxygen per hour into the aeration tank. To maintain aerobic conditions or a particular level of dissolved oxygen at least 7.96 lb. of this 21.8 lb. of oxygen must go into solution. This means an oxygen transfer efficiency of 36.5 per cent under the conditions described. Compared to standard aeration with a transfer efficiency of about 5 per cent this is a high value.

A. A. Kalinske (6) reported recently that newly developed mechanical aerators using compressed air also show oxygen transfer efficiencies of 30 to 40 per cent. Due to their high power requirements, however, an economical operating range of 10 to 12 per cent transfer efficiency was recommended. The data presented in this paper show that it is possible to operate at higher transfer efficiencies without using excessive power. Unfortunately, due to lack of equipment, no direct determinations of the oxygen utilization rate could be made in the field. It is hoped that future studies will provide additional data with respect to this interesting problem, particularly the relationship between oxygen utilization rate and loading.

Biological Load

The load applied per pound of mixed liquor suspended solids per day is the biological loading factor and is an important consideration in any waste treatment plant. As previously mentioned, the mixed liquor carried an average of 556 lb. of suspended solids at an average B.O.D. load of 212 lb. per day. The average biological loading factor is therefore 0.374 lb. of B.O.D. per pound of suspended solids. The highest B.O.D. load during the test period was 390 lb. at 2,950 p.p.m. suspended solids. This means that the loading factor increased to 0.8 lb. of B.O.D. per pound of suspended solids per day. The normal range used in activated sludge treatment is 0.25 to 0.5 lb. of B.O.D. per pound of suspended solids per day.

Eckenfelder (7) has shown, for different types of organic wastes, that up to a B.O.D. load of 1 lb. per day per pound of suspended solids a B.O.D. removal of 90 per cent can be obtained. According to these results it should be possible to apply a maximum load of 556 lb. of B.O.D. per day at the average level of 3,420 p.p.m. suspended solids, provided the detention time or the oxygen supply does not become a limiting factor. Apparently at an air delivery of 20 c.f.m. the latter is the case. Even though the equipment was able to maintain the necessary amount of suspended solids in an aerobic condition under a load of 200 to 300 lb. of B.O.D. per day, this would not be possible under a load of 556 lb. of B.O.D. due to the higher oxygen utilization rates occurring under these circumstances. According to other work by Eckenfelder (8), the oxygen utilization rate closely follows the B.O.D. loading. The expected B.O.D. removal would be 510 lb. per day. Using the previous value of 110 cu. ft. of air per lb. of B.O.D. removed per day, an air supply of 39 c.f.m. would be necessary.

Acknowledgment

The data used in this paper were collected while the author was Re-

search Director, International Agricultural Products, Inc., Green Bay, Wis.

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REPORT OF THE NTTCIW MEETING, DECEMBER, 1955

The National Technical Task Committee on Industrial Wastes (NTTCIW) heard task group reports and learned of the need for detailed research into basic industrial waste problems during its meeting at the Robert A. Taft Sanitary Engineering Center, Cincinnati, Ohio on December 1–2, 1955. The committee was formed in 1950 to cooperate with the USPHS on technical control of water pollution by industrial wastes, as authorized by Public Law 845. Officers elected for 1956 were as follows: Dr. C. Fred Gurnham, Chairman; Frank J. McKee, Vice-Chairman; Louis F. Warrick, Secretary.

The need for more research personnel on basic water pollution problems was emphasized by Harry A. Faber, Research Grants Officer, Division of Sanitary Engineering Services, USPHS. Although funds are available, grants have often been withheld for lack of qualified personnel. Only two per cent of the estimated 5,000 practicing sanitary engineers in the United States are engaged in research. Activities of the research subcommittee are being directed at what industry can do to relieve the personnel shortage.

The general sessions centered on problems of mutual interest to the task groups. R. W. Hess, National Aniline Division, discussed some industrial wastes problems needing research. K. S. Watson, General Electric Co., urged more attention to the treatment of industrial wastes in municipally-owned facilities, including pretreatment by industries. The dangers of writing water quality criteria into pollution control laws were stressed by A. J. Steffen, Wilson & Co. The program of the Taft Sanitary Engineering Center in providing a sound basis for water quality criteria, was explained by Dr. Berger, USPHS. Gordon E. McCallum briefed the committee on the revised water pollution bill pending before Congress.

The four task groups of the committee reported on their accomplishments and proposals for the abatement and prevention of water pollution. The food industries group report on the reduction of wastes through by-product recovery, water re-use and in-plant control. Spray irrigation and aerobic digestion are being emphasized. The mineral products group reported on improvements in waste treatment equipment and operations. Progress was being made in research and the preparation of manuals for control of waste water. The chemical processing group listed research work on detergent pollution as one of its principal activities. Special research efforts of the paper, tanning, electroplating and textile industries were described. The general industry group indicated progress in its respective industries, such as automobile, electrical and power, and the railroads.

WINSTON-SALEM SEWER EXPANSION PROGRAM *

By P. D. Davis

Consulting Engineer, Platt and Davis, Engineers, Durham, N. C.

The proper development of plans for any sewage program, particularly where sewage treatment is involved, requires considerable detailed study and investigation before the actual preparation of plans can be started. It would be rare, indeed, to find two communities in which the water-borne wastes had the some characteristics. Average values of pH, color, B.O.D. and quantity frequently mean little to the designer. The pH may vary from 4 to 12, color from that of dishwater to dark brown, blue, red or black; B.O.D. from less than 100 to more than 2,000 p.p.m.; and the average flow of 2 m.g.d. might be as low as 200,000 g.p.d. or as high as 10 m.g.d. during a 24-hr. period.

It is not infrequent that the wastes from some water users are not at present tributary to the sewer system, but should be connected if an over-all clean-up program is to be effective. The addition of such wastes sometimes overloads existing sewers and changes the general character of the sewage reaching the treatment plant site. The engineer designing sewers 40 years ago could not foresee that the per capita consumption of water would increase from 40 g.p.d. to 150 g.p.d. or more and that industry of undreamed-of variety would require many additional thousands of gallons of water, the waste from which would add to the load in the sewer line and at the treatment plant.

Winston-Salem is heir to all of the previously mentioned problems and some not referred to. Its population has grown from 13,600 in 1900, to about 88,000 in 1950, at an average compounded annual rate of 3.79 per cent. In addition, there is a large fringe population just outside the corporate limits, of which the city is now serving some 7 to 10,000 persons. It is estimated that the 1975 population of Winston-Salem will be approximately 160,000. Its principal industries use more than 2.5 m.g. of water per day, of which about one-half now enters the sewers. The combined B.O.D. of the waste from these industries has a population equivalent of 70,000.

Sewer System

The Winston-Salem area drains to several small creeks, all tributary to Muddy Creek, which in turn is a part of the Yadkin River system. In determining sewage flows, drainage maps were prepared showing each of the main drainage areas in and adjacent to the city. Each main drainage area was subdivided into small areas either now served or to be served in the future by lateral sewers tributary to the main trunk sewers. Each area was numbered, and its acreage determined. Ultimate flow calculations were based on an over-all population of 12 persons per acre, a per capita consumption of 150 g.p.d. and a discharge rate of 150 per cent. Each acre contributes 1,800 g.p.d. at a maximum rate of 2,700 g.p.d.

Park and other areas not suitable for development were excluded. A tabulation of total required capacity of each main trunk sewer was then prepared with industrial waste increments added

* Presented at 1955 Annual Meeting, North Carolina Sewage and Industrial Waste Assn.; Winston-Salem, N. C.; Nov. 14–16, 1955.

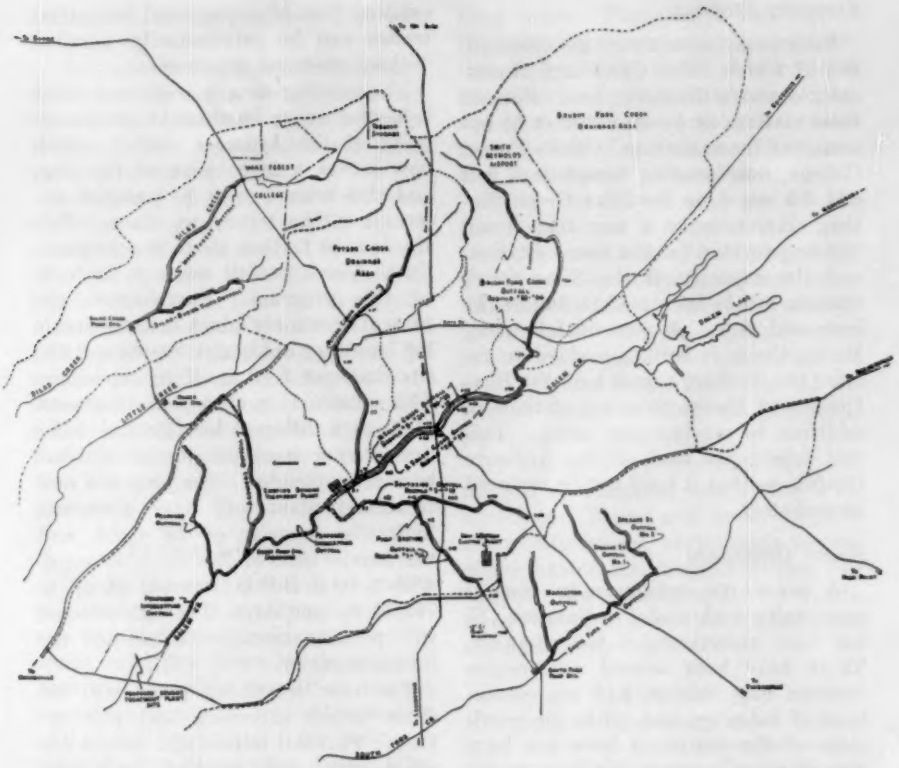

FIGURE 1.—Existing and proposed intercepting sewer outfalls, Winston-Salem, N. C.

at the proper points. The grade, size and capacity of existing sewers were determined, either from field surveys or existing profiles and maps, and the results tabulated along with the required capacity. All existing main sewers were inspected, and several were found to be undercapacity and overflowing during peak flow periods.

Interceptors

Based on the study of the sewer system, recommendations were made for new or replaced interceptor sewers as shown on the drainage map, Figure 1. These recommendations were:

1. The North Salem Creek outfall sewer should be replaced with 48-in. pipe from Peters Creek to Brushy Fork Creek.

2. A new outfall sewer should be constructed along Burke Creek, using 18- and 24-in. pipe, and the discharge from the Hanes Pumping Station and the Silas Creek Pumping Station should be diverted to this outfall to relieve the overload condition of the existing Ardmore Outfall.

3. The southside or Fogle Creek Outfall should be replaced with 27- and 24-in. pipe to a point near the Western Electric Company factory.

4. The Brushy Fork Creek Outfall should be replaced with 30- and 36-in. pipe to the vicinity of the airport.

5. A new 54- to 60-in. main interceptor should be constructed along Salem Creek from Peters Creek to the site of the new sewage treatment plant.

Pumping Stations

Four pumping stations are operated, two of which, Silas Creek and Hanes, use a common discharge line. Both of these stations at present cannot be operated at the same time. Wake Forest College, now nearing completion, will add 0.5 m.g.d. to the Silas Creek Station. Accordingly, a new force main will be provided for the Hanes Station, and the capacity of the Silas Creek Station will be increased to handle the increased flow. A new outfall along Burke Creek is being provided to receive the discharge from both the Silas Creek and Hanes pumping stations in addition to serving new areas. This will relieve the load on the Ardmore Outfall, so that it need not be replaced immediately.

Sewer Ordinance

A sewer use ordinance in general conformity with model ordinances (1) has been recommended for adoption. There have been several conferences between city officials and representatives of industry, and, while all provisions of the ordinance have not been agreed upon, progress has been made. Since there will be an excess capacity in the early years of operation of the treatment plant, it has been recommended that the final provisions of the ordinance not be adopted until after all industries are connected to the sewers and there has been sufficient operating experience to determine the effect of the composite flow of wastes and sewage on the treatment processes.

Sewage Treatment Plant

Pilot plant studies during 1951 indicate that high-rate trickling filters will provide satisfactory secondary treatment. Pilot plant studies with vacuators indicate a removal of 30 to 35 per cent suspended solids and 20 to 25 per cent B.O.D. when used in place of primary sedimentation units. These studies also indicate that the existing flow of sewage and industrial wastes can be satisfactorily handled without chemical treatment.

The existing sewage treatment plant is located two miles above the discharge point of the Ardmore outfall, which now serves a large part of the city, and this sewage must be pumped upstream to the treatment plant. This site is even farther above the proposed Burke Creek outfall sewer to be built by this program. Furthermore the present treatment plant is not suitable for handling industrial wastes and the site does not lend itself to expansion. The construction of a new treatment plant at a different location five miles downstream from the present site has been recommended. The proposed new treatment plant will have a normal hydraulic capacity of 25 m.g.d. and will handle rates of flow up to 36 m.g.d. with a total B.O.D. removal of up to 75,000 lb. per day. The site selected will permit expansion to double the initial capacity.

The new treatment plant will include coarse screening and grit removal, Parshall flumes and meters for measurement and control, twin vacuators, four high-rate trickling filters, final settling, and chlorination of the final effluent. Three heated and 1 unheated separate sludge digestion tanks and six acres of sludge drying beds are being provided for handling the sludge. Digester gas will be utilized for power and heat.

Construction Schedule

The total cost of the improvements recommended was estimated in the preliminary report at approximately $6.5 million. The plans have been accepted by the city and the necessary funds for construction have been authorized.

Contracts have been awarded for construction of the Peters Creek interceptor. The interior of the reinforced concrete pipe will have a bitumastic

coating and rubber gaskets will be used for jointing.

It is anticipated that approximately two years will be required to construct the sewage treatment plant. Equipment contracts specify delivery dates starting in October, 1956.

Additional contracts for the construction of sewer lines will not be awarded until near the end of the construction period for the treatment plant. These sewer lines will serve populated areas and it will be necessary to pick up the tributary sewage from the smaller outfalls as interceptor construction reaches them. Construction schedules will be arranged so that these lines can be completed about the same time the treatment plant is placed in operation.

Proposed Operation

During the early years of treatment plant operation it is proposed to operate the high-rate trickling filters in series, using two filters for the primary pass and the other two filters for the secondary pass. A recirculation rate of 1:1 will be provided for each pass through the filters, the recirculated liquor being taken from the final settling tanks. Two pumps will deliver the incoming sewage and the recirculation liquor to the first two filters and two other pumps will deliver to the second pair of filters. Arrangements are being provided so that the filters can also be operated in parallel.

When the initial capacity of the treatment plant is reached it is proposed to operate the filters in parallel and provide a third stage of treatment. This will be either activated sludge or additional trickling filters, the decision to be made based on experience in operating the initial facilities. If activated sludge is used, no additional pumping will be required.

It has been recommended that the number of personnel now in the Winston-Salem Water and Sewer Department be increased, particularly for engineering and maintenance service. In addition, the salaries and work conditions of treatment plant operators should be reviewed.

Reference

1. "Municipal Sewer Ordinances." Manual of Practice No. 3, Federation of Sewage Works Assns., Champaign, Ill. (1949).

FEDERATION MEETING PLANS

Response of the members to the suggestion of a special convention train to the Los Angeles Meeting indicates the favorable reception of such a plan. The New York Member Association is working locally to arouse interest in making up several cars for a special train. The New York group would meet with others in Chicago for a Federation train to California.

It is not too early to plan a different vacation for you and your family—an Autumn Vacation in Southern California. The Federation Secretary is very interested in your individual and collective plans. Send him a postcard today with your comments.

Remember Los Angeles, October 8–11, 1956

SEWAGE PUMPING STATION DESIGN PRINCIPLES *

By Raymond G. Martin

Civil Engineering Associate, City of Los Angeles, Los Angeles, Calif.†

In almost every new sewer system or expansion of an existing system, it is necessary to provide means of lifting the sewage at some point. In areas where the topography is flat or contains depressions, it is often less expensive to provide pumping facilities than to construct gravity sewers in deep excavations or through tunnels.

Expansion of urban areas beyond existing sewer facilities often necessitates the use of pumping stations to discharge sewage into existing sewers or treatment plants. It is sometimes necessary when sewage treatment plants are constructed in areas having a relatively flat topography to pump the sewage at the treatment plant site to obtain gravity flow through the plant. Regardless of the reason for their use pumping stations are less desirable than gravity flow. Every pumping station has maintenance and operational problems. Operating personnel and power are continuing cost items. Therefore, a sewage pumping station should never be constructed where a safe, economical substitute will suffice.

Types of Pumping Stations

A sewage pumping station is a collection of equipment and materials assembled for but one purpose—to pump sewage.

There are many types of pumping stations in existence today which may be classified as follows:

1. Wet pit pumping stations consist of a sump or wet well with the pump or pumps submerged in the sewage. In this type station the pump, shaft and supporting facilities are exposed to the sewage. The motor is usually located at an elevation above the pump and in most cases is exposed to an atmosphere of sewage gases. The difference in elevation of the pump and the motor requires a long shaft, which frequently is a source of trouble. The present trend is to avoid wet pit type construction.

2. Pneumatic ejector pumping stations may be used to great advantage for small installations. An ejector consists of a large pot which receives the sewage and expels it by means of compressed air. Despite their low efficiency (15 per cent) ejectors are desirable for small installations because little maintenance is required.

3. Dry well pumping stations are the most generally used type. The pumps, motors and all appurtenant equipment are isolated from the sewage and located in a dry, clean area. Dry well pumping may be practiced using: (a) constant speed pumps, provided there is storage to take care of the variations in flow; (b) variable speed pumps with little or no storage.

The second method is relatively new and is not widely used at the present time. Where constant speed pumps are used approximately one-half of the station structure is designed for the storage of sewage. This portion of the station requires hard and dirty work to keep it clean and functioning properly. As a result maintenance personnel may neglect this work, which frequently causes operating trouble.

* Presented at 1955 Annual Meeting, California Sewage and Industrial Wastes Assn.; Riverside, Calif.; April 27–30, 1955.

† Presently, City Engineer, Santa Barbara, Calif.

In comparing the two methods of dry well construction, it is apparent that capital costs are much greater for construction of the storage type structure. This is not only true because of the area requirement, but more so because of the additional depth required for storage. In either case, the receiving facilities are equal for equal flow. Reduction of the storage areas required does not eliminate all maintenance; however, it does reduce the quantity and some of the objectionable portions of the maintenance work.

It is not the scope of this paper to discuss all the design characteristics of a sewage pumping station. A few general features and procedures are included and some design problems are discussed in detail. However, it is a responsibility of the design engineer to consider in detail every feature, piece of equipment and operation of the entire pumping station. This includes everything from the dynamics of the pump impeller to the color of the paint on the walls.

Design Considerations

The first phase of pumping station design is to make a preliminary investigation to determine if a pumping station is necessary. Many troublesome pumping facilities could have been eliminated if their necessity had been kept in mind. Economics is not the only consideration relative to feasibility of design. There are many intangibles connected with pumping stations which must also be considered.

Location

Location of the pumping station is relatively simple in new developments, but in the expansion and remodeling of an existing system it may require extensive study. Both the anticipated and existing flow must be determined, since frequently the failure of a station can be traced to an incorrect analysis of the flow. Proximity to residences and possible odor and noise nuisances must also be considered.

Finally the type of pumping station must be determined. Here, the engineer's background and ability are most important. Consideration must be given to a number of factors including: (a) the qualifications of the available maintenance personnel; (b) the sewer system as a whole; and (c) the type and quality of sewage to be handled.

Flow

In studying the flow there are two primary characteristics to be considered: (a) the flow duration curve (Figure 1) or the rate of flow relative to

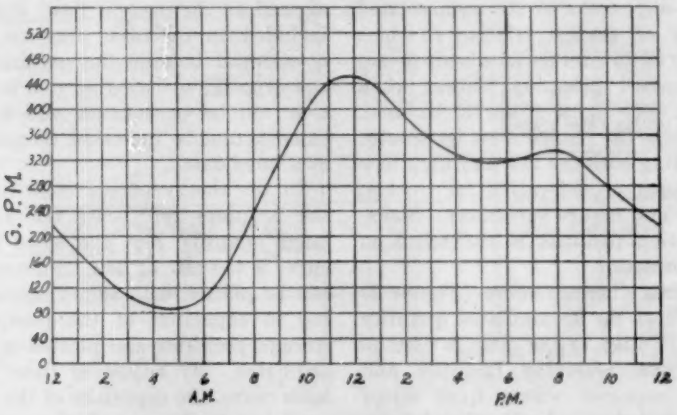

FIGURE 1.—Typical flow duration curve for sanitary sewage.

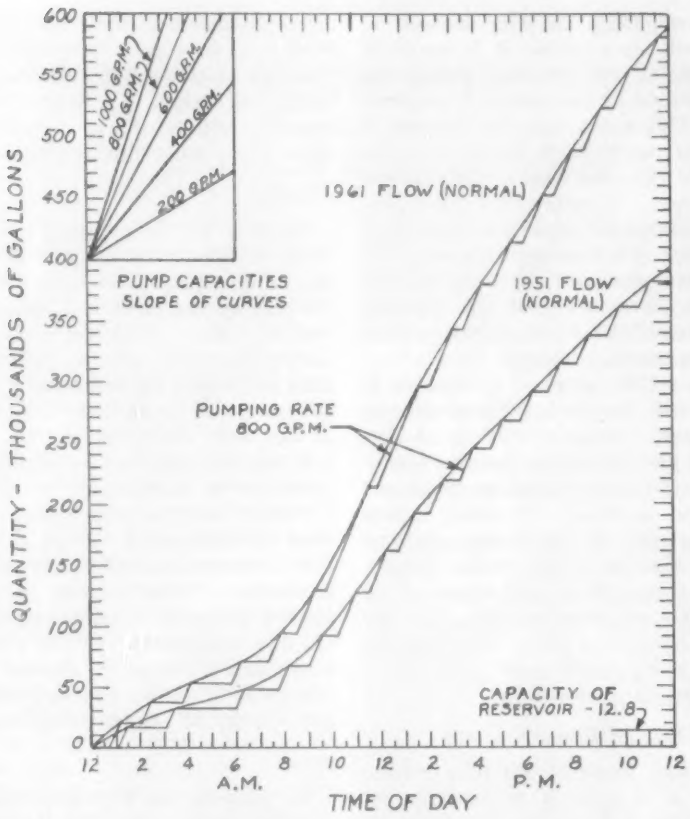

FIGURE 2.—Typical mass diagram for a wet well, illustrating the operation of the pumps.

time of day, and (b) the accumulated quantity of sewage relative to time. The rate of flow serves as a basis to size the proposed pumping station when variable speed pumps are to be used. In this case the pumps must be capable of handling both the low and high flow with proper consideration for weekly, seasonal and future variations. Sizing of receiving facilities is also based on this information.

The mass diagram curve (Figure 2) which shows the accumulated quantity of flow relative to the time is used to size pumps, receiving facilities and storage required where fixed speed pumps are to be used. Predicted flows, as well as the present flow, should be included on the mass diagram. This is essential because the required storage capacity as based on this information will be permanent, and in most cases cannot be increased economically at a later date.

In the theory of the mass diagram the ordinate represents the accumulated quantity and the slope of the curve is the rate of flow. Pump curves can be plotted with slopes corresponding to capacities of the pumps and storage facilities are plotted as fixed ordinates. By adjusting these to the mass curve, the capacities of the pump, receiving facilities and storage reser-

voir can be determined for any desired time. This analysis presents a picture of the pump cycles of the pumping station operation. For small flows the mass diagram principle also may be applied to variable speed pump design. However, in large rated flow pumping stations the storage element is negligible, making this method impractical.

Sizing of Pumps

There are two methods of preparing plans and specifications for pumps. The first is to ask a pump manufacturer to furnish a pump for the installation. The second is to develop the design and to specify the conditions to be met by the pump. The first method eliminates competition and may have an adverse effect on the quality of equipment procured. This does not mean that the design could not be developed around a particular pump. Quality and performance, should be the basis for selection rather than brand name. Close cooperation between the pump manufacturers and the engineer is essential for good design.

Pump Characteristics

Figure 3 is a typical pump analysis for operation only, showing the system curves and the pump characteristics curve. This particular analysis was not for a new installation but for an expansion of an existing facility. This pumping station was operating at a maximum capacity of 32 c.f.s. and it was necessary to provide for an increase in capacity. The total head on the pumps was 90 ft. This particular curve shows four different system curves and one force main curve. The upper system curves represent the total loss in the system and are equal to the sum of the piping losses and the force main loss. The four different curves represent four possible operations, such as 1, 2, 3 or 4 pumps operating together. This is known as compounding pumps in parallel. The wide separation between the force main friction curve and the total system curve indicates a considerable length of pump manifold. While this is not desirable, there was no way of eliminating this condition.

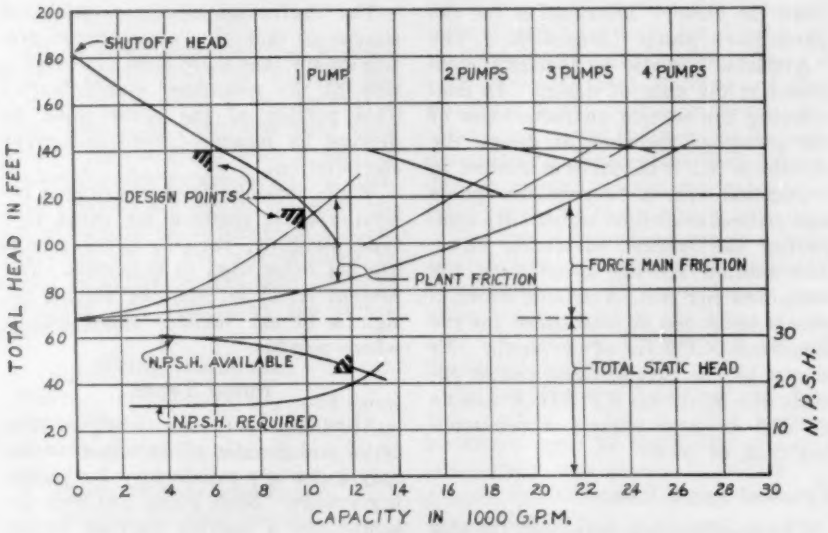

FIGURE 3.—Typical pump analysis curves showing the system and characteristic curves.

Also shown are four pump curves which represent any combination of the four pumps compounded in parallel. The point of intersection of the pump curve and the system curve is the point at which the pumping station would operate if all other conditions remained equal. However, in actual practice this is not true. The system curve varies with a change in friction coefficient as well as changes in the water level of the wet well. Therefore, operation can be predicted within a range but not to a point. In almost all cases it is advisable to analyze the pumps for maximum and net average static head.

The design points for the particular pumps being studied are at 10,000 g.p.m. and 6,000 g.p.m. In the lower left hand corner are shown two curves labeled N.P.S.H. Available and N.P.S.H. Required. The term N.P.S.H. denotes the net positive suction head and is defined as the gage reading in feet taken on the suction nozzle as referred to the pump centerline, minus the gage vapor pressure in feet corresponding to the temperature of the liquid, plus the velocity head at this point.

The "Required" curve was plotted from the factory information for the particular pump furnished. The "Available" curve is normally computed at the time of design. In considering the suction characteristics of the pump at the time of design the Available N.P.S.H. curve is studied in connection with suction specific speeds and critical cavitation sigma. By comparing the critical cavitation sigma and suction specific speed with the computed N.P.S.H. Available curve, a design point can be established for the Required N.P.S.H. of a pump. As shown by the curves, these pumps operate at a minimum N.P.S.H. Available of 22.5 ft. and require a minimum N.P.S.H. of 20 ft.

Constant Speed Pumps

The specifications developed for this particular pump relate to characteristics only. The characteristics of the pump shall provide 10,000 g.p.m. at 115 ft. total dynamic head and 6,000 g.p.m. at 140 ft. Variation in head shall not exceed + 2 per cent or − 4 per cent. Maximum brake horsepower shall not exceed 400 at capacities from 0 to 12,000 g.p.m. Required N.P.S.H. shall not exceed 22.5 ft. at 12,000 g.p.m. and the pump shall not cavitate under this condition.

Variable Speed Pumps

The design is somewhat different when variable speed pumps are used. Figure 4 shows the analysis of a two-pump variable speed installation. The head capacity curves are plotted for various speeds of pump operation. In addition, the system curves are shown for one pump and for both pumps in operation. The system curve is not continuous because the first pump must be operating at full speed before the second pump cuts in. When a horizontal line is plotted from any point on the system curve the intersections of this line with the characteristics curves indicate the operation of each pump for a given capacity.

The derivation of these curves is similar to that of a conventional system except that there is no direct solution to the secondary system curve. This portion of the curve must be derived by means of trial and error computations.

While variable speed pumping is becoming quite common for rated flow type pumping, variable speed pumps are not often used in multiples. The present trend in pumping station design is to use variable speed pumps where possible.

Pump Types

Although there are many makes, types and designs of pumps available only a few are satisfactory for pumping sewage. Each pump has been designed for a specific purpose within limiting conditions and to use a pump

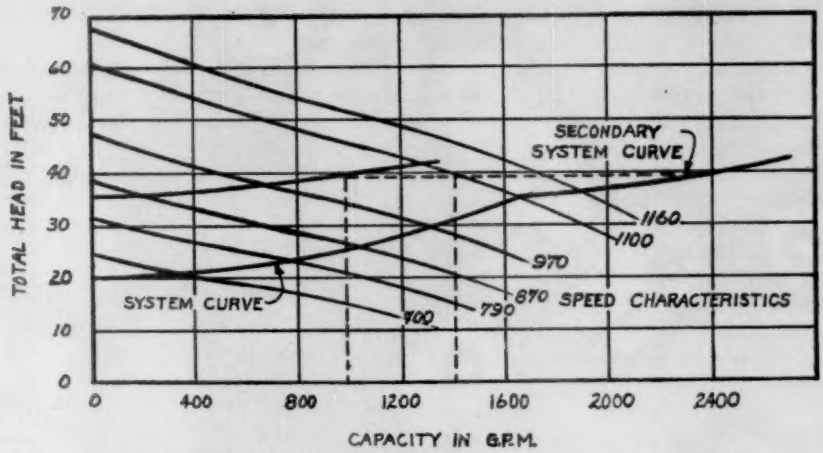

FIGURE 4.—Typical analysis of a two-pump variable speed installation.

for any situation for which it was not designed is certain to cause trouble.

Where variable speed design is used the following methods have proven satisfactory for obtaining the desired results:

1. Wound rotor motors.
2. Hydraulic couplings.
3. Variable pitch pullies with belt drives.
4. Magnetic couplings.

Each of these methods gives approximately the same results, although the first method cannot be changed at a later date. Another relatively new method uses a pump with a built-in bypass for the return of excess flow.

The pumps are one of the most important parts of a pumping station. Therefore, it is not unreasonable to spend considerable time and effort in designing and specifying the equipment.

Pump Installation

Sewage pumps in a dry well type pumping station may be installed by one of the following methods:

1. Pumps with isolated motors.
2. Pedestal mounted pumps.
3. Close coupled pumps.
4. Horizontal pumps.

Methods 1, 2 and 4 may be used for installations of variable speed pumps.

The use of isolated motors is the oldest method and is still the most common. The motor is isolated by placing it above the highest possible flooding level, which is usually at the ground line. The pump is then connected to the motor with a long shaft which, when not installed perfectly, is a constant source of trouble and expense. During recent years the trend has been to eliminate this long shaft. Experience indicates that flooding of the pump pit is not caused by storms, accidents and power failures, but by poor design and maintenance. With proper design, isolated motors are not essential for reliable operation.

The use of horizontal pumps is considered best for ease of maintenance and installation. However, larger floor areas are required in the pump room and this adds to the capital cost of the pumping station.

To complete the design of the dry well portion of the pumping station provision must be made for all supplementary equipment necessary to proper operation and maintenance of the station. This equipment may be classified according to the following groups:

1. The piping system includes valves and supports, taps for gages, drains and flexible couplings. Substantial bases and flexible couplings should be provided to prevent transmission of pump vibrations. Flexible rubber fittings are now available which have sufficient strength and life to be satisfactory for sewage pumping.

2. A priming system for the pumps is essential where continuous automatic operation is intended. This may be accomplished conveniently without the use of special priming equipment if proper consideration is given to operation and arrangement of equipment. Interconnecting the pumps in priming and air release systems must be avoided.

3. Access to the equipment should be given careful consideration since it is customary for an operator to service the station alone. All hazards should be removed by design and not by experience. Hatches and doors must be sized for the removal of all equipment and hoisting equipment provided for the handling of heavy or massive equipment.

Pump Controls

Where automatic operation is proposed the heart of the pumping station is the controls. If the controls fail to respond to flow or liquid level changes the entire station fails. Among the many types of controls presently available are the electrodes, the floats, the air bubbling systems, the electronic and radar systems, and the flow meters.

All controls require some maintenance for continuous satisfactory operation. Normal sewage contains solids, grease and rags, which cause clogging, shorting and fouling of the controls. To keep the controls in operation it is necessary to clean them frequently and in some locations this may be as often as twice a day. Cleaning and servicing controls is a dirty and expensive job. Therefore, every consideration should be given to the design and selection of controls in an effort to eliminate cleaning costs.

The electrical system of a pumping station is similar to that of any industrial installation, except that portions of a pumping station are often subjected to hazardous conditions caused by the sewage atmosphere. In some locations this requires the use of explosion-proof fixtures.

Receiving Facilities

The receiving portion of a pumping station consists of a structure and equipment to receive, control, screen and store the sewage. In a rated flow type pumping station little or no storage is provided. With this one exception, the required receiving facilities are the same for either type of dry well construction.

The pumping station, depending on size, should be provided with a control gate, sand trap, diversion gates, bar screen and either a grinder or a comminutor. In addition, pump suction connections, stilling basin and hose bibbs for cleaning should be incorporated in the design.

The inlet sewer should always be provided with a gate valve or sluice gate to stop the flow of sewage for reasons of maintenance and emergency. Frequently this control is located in a manhole near the pumping station. When possible the inlet sewer should be provided with an overflow or emergency bypass.

When the pumping station is of sufficient size the inclusion of a sand trap may be warranted. Normally a 6-in. to 8-in. depression in the influent channel is sufficient. This prevents the accumulation of sand in the wet well where it is difficult to remove.

Bar screens are necessary regardless of the use of automatic grinders. Provisions must be made for cleaning these screens and removing the screenings from the station. When an automatic grinder is provided the inlet channel is split following the sand trap to pro-

vide for both the grinder and a bar screen. Experience shows that grease is now beginning to cause trouble with grinders. Manufacturers of this equipment should be consulted concerning the particular features of their units.

Sewage from the bar screen and grinder should not discharge so that it causes violent turbulance in the storage area. This turbulance is undesirable since it causes the liberation of dissolved gases from the sewage, particularly hydrogen sulfide.

The stilling basin or storage reservoir is where the sewage is stored for constant speed pumping and where it is allowed to become quiescent before pumping in a rated flow pumping station. It is also in this area that the pump controls are located.

Ventilation is essential before any person can enter an active wet well. In many installations ventilation is a constant problem. If the wet well is not ventilated it is unsafe for operators to enter; if it is ventilated local residents complain of odor nuisance. Recent tests made by the city of Los Angeles, Calif. indicate that a 5 min. air change is both sufficient and desirable, provided the atmosphere entering through the inlet sewer is included in the quantity to be exhausted.

Other Considerations

Once the capacity and characteristics of the pumps have been determined, all other hydraulic features of the station can be designed. The force main is designed as a pressure pipe based on pump capacity. Air pockets either should be avoided or provision made for the release of air or trapped gases.

The appearance of the above ground pumping station structure and the landscaping of the grounds should not be neglected. Structures and landscaping that blend with the contemporary features of the surrounding area are essential if the pumping station is to be accepted in the neighborhood. For large pumping facilities provision should be made for operating personnel and visitors.

Pumping stations are normally designed for a life of 40 to 50 years. With good engineering design they should satisfactorily serve for this period.

PROFESSIONAL OPPORTUNITY

The newly reorganized sanitary engineering program at Alabama Polytechnic Institute provides a professional opportunity in education. At present a faculty position in Sanitary Engineering is available for an assistant or associate professor to teach undergraduate courses in water and sewage theory and design. Research work in these and related fields will be carried on by the faculty.

Requirements include a Master of Science degree in Sanitary Engineering or Civil Engineering with a major in sanitary work. Teaching experience is desirable. Salary is open, depending on qualifications.

Information may be obtained by writing to Professor Theodore Jaffe, Civil Engineering Department, Alabama Polytechnic Institute, Auburn, Ala.

SEWAGE SLUDGE THICKENING TESTS *

By Sterling G. Brisbin

Development Engineer, Sanitary Engineering Technical Division, Dorr-Oliver Inc., Stamford, Conn.

Twenty-four years ago an editorial appeared in the *Sewage Works Journal* (1) which included the statement: "Another marked improvement in the digestion of sludge would be available if fresh solids could be so concentrated before entering the digester tank that it would be unnecessary to draw-off supernatant liquor."

Prior to and since this statement was made design engineers and sewage treatment plant operators have striven for this goal. Inert or stable materials have been concentrated in thickeners by industry for years. However, sewage sludge presents a special problem: the sludges are unstable and decomposition occurs during the time required for thickening (concentration). The decomposition produces gas which adheres to the solid particles, changes their densities and tends to buoy them to the surface of the liquor.

Various methods have been used in an attempt to delay decomposition. In 1934 at Phoenix, Arizona, additions of chlorine and lime were used with varying degrees of success (2). Another method of thickening utilized the effect of decomposition and allowed septic action to cause flotation of solids. After flotation, a subnatant was drawn off and the solids remained in a concentrated state (3). In 1953 a method was developed by Torpey † to thicken sewage sludge on a continuous basis without the addition of chemicals.

Generally, the flow pattern permits dilute sludge from the primary clarifiers alone or combined with sludge from the secondary clarifiers to be fed to the center feedwell of the thickener. The solids settle, thicken in a definite "blanket" zone and are drawn from the bottom of the tank. The excess liquid, which includes the portion which would ordinarily be the supernatant from digested sludge, is decanted by a peripheral weir.

The degree to which sewage sludges can be thickened depends on several factors. One important factor that determines the solids concentration obtainable is the source of the sludge, whether it comes from primary treatment, trickling filters, activated sludge or other process.

The work by Torpey with combined primary and activated sludges has been reported previously (4). Subsequent tests were conducted with primary sludge alone (5). In order to obtain operating and design data for the thickening of combined primary and trickling filter sludges, the sewage treatment plant at Middleborough, Mass. was chosen as a site for a test installation.

Test Installation

The town population of Middleborough approximates 8,000, but it is incompletely sewered and only 2,500 people are served by a total of 700 connections. The sewage system is composed of combined sanitary and storm sewers and the treatment plant was designed for a flow of 2.0 m.g.d. The flow diagram was modified to include the sludge

* Presented at Fall Meeting, New England Sewage and Industrial Wastes Assn.; Woodland Country Club, Auburndale, Mass.; Nov. 3, 1955.

† Densludge Thickening Process, Dorr-Oliver Inc.

FIGURE 1.—Experimental sludge thickener.

thickener (Figure 1). An existing plunger type sludge pump was used to pump primary sludge to the thickener, and an existing centrifugal pump, which originally pumped the secondary sludge to the primary clarifier, was used to divert the secondary sludge to the thickener. These two pumps, as well as the thickened sludge-withdrawal pump, were operated by time clocks because the pump capacities were too great for continuous operation. The thickener was 7 ft. in diameter, with a sidewater depth of 11 ft. and handled all the sludge collected. Taps were provided in the tank wall to permit sampling at various depths.

The thickener overflowed by gravity through a flexible plastic pipe and a recording weir box to the primary clarifier effluent. Ordinarily, this overflow would be returned to the primary clarifier influent if feasible, but the hydraulics did not permit such an arrangement for this temporary installation. Thickened sludge could be pumped to the existing digester, an experimental digester, or it could be divided and a portion sent to each digester.

Test Periods

Extensive tests were started on August 13, 1954. During the first 30 days of operation, the flow to the treatment plant was abnormally low due to a clogged sewer at a bypass to the river. Late in September, power failures due to hurricanes caused a few days of interruption in operation. Operations proceeded until December 1 when the freezing of exposed piping on the sludge thickener forced termination of the project for the remainder of the winter season.

Test operations were resumed May 13, 1955 and it was soon evident that the treatment plant loading conditions had changed. The solids loading had increased from an average of 300 lb. per day to 760 lb. per day. This was due partly to 40 new sewer connections added in the early spring. However, most of the additional solids could be attributed to abnormally large volumes of septic tank contents that were being discharged to the system. The added sludge from the septic tanks contained such a high percentage of digested material (often as much as 50 per cent), that it was not representative of normal sewage. With this condition, valid information on the thickening of combined primary and trickling filter (secondary) sludge could not be obtained. Although the thickener was apparently capable of handling effectively the increased loading, temporary diversion of septic tank cleanings was requested for the remainder of the test period—June through August, 1955.

Test Data

Data for the thickener operation (Table I) is self-explanatory and is

TABLE I.—Sludge Thickener Data, Middleborough, Mass.

Period	Flow (m.g.d.)	Influent						Underflow					Overflow		
		Temp. (° F.)	Solids Ratio, Secondary	Liquid Ratio, Secondary	Total Solids		gal./sq. ft./day	Total Solids (%)	Volatile Solids (%)	Flow (gal./day)	Flow (gal./mg. raw sewage)	Total Solids (lb./day)	Susp. Solids (p.p.m.)	Solids Recovered (%)	Sludge Vol. Ratio (days)
					lb./day	(lb./sq. ft./day)									
1954 Aug. 13 to Sept. 12	0.200	65	.27	8.1	160	4.2	750	6.0	75.0	337	1680	142	100	90	—
Sept. 13 to Sept. 20	0.535	63	—	8.0	320	8.3	710	9.4	77.6	322	600	286	120	90	3.2
Sept. 21 to Sept. 31	0.674	62	—	8.0	292	7.6	710	9.5	79.0	410	735	260	135	91	2.3
Oct. 1 to Oct. 31	0.416	61	—	8.0	300	7.8	710	7.35	80.3	391	940	265	130	92	2.2
Nov. 1 to Nov. 30	0.543	54	—	8.0	260	6.8	710	7.50	80.4	378	700	232	130	90	2.1
1955 May 13 to May 31	0.523	56	—	8.0	760	19.8	730	8.7	58.0	930	2100	985	—	90[1]	1.0
June 1 to June 30	0.425	60	—	8.0	455	11.8	780	10.3	66.0	480	1120	400	155	90	1.5
July 1 to July 31	0.322	63	.16	8.0	473	12.3	780	8.3	67.5	620	1800	425	145	88	1.3
Aug. 1 to Aug. 31	0.426	65	.27	8.0	455	11.8	780	8.4	63.3	490	1300	410	—	90[1]	1.2
Average	0.452	60.8	.24	8.0	385	10.0	730	8.37	71.8	484	1215	345	115	90	1.7

[1] Estimated.

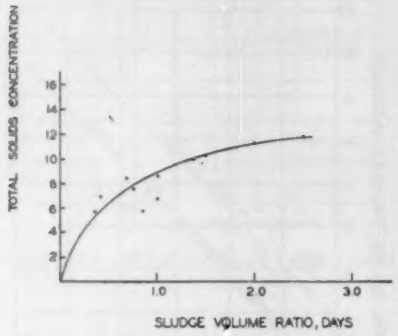

FIGURE 2.—Sludge volume ratio versus total solids concentration.

divided into monthly periods where possible. These periods are further subdivided only when the flow or other operating conditions changed drastically. The liquid ratio reported is the proportion of secondary sludge volume to primary sludge volume and averaged 8.0.

The sludge volume ratio reported is the same as that defined by Torpey (4) as "the volume of sludge blanket divided by the daily volume of sludge pumped from the thickener." A graph of the total solids concentration versus the sludge volume ratio is shown in Figure 2 as obtained from results at Middleborough. This graph agrees closely with data presented by Torpey. Initially a small increase in solid volume ratio produces a fairly rapid increase in sludge concentration. For a sludge volume ratio greater than one, the increase in concentration is less rapid per unit increase in the ratio.

The length of time that sludge may be held for concentration, as measured by the sludge volume ratio, is determined primarily by its freshness and the speed with which septicity may occur. These in turn are affected by factors such as the length and slope of the sewer outfall, and the temperature of the sewage.

FIGURE 3.—Thickened sludge with a 15 per cent solids concentration.

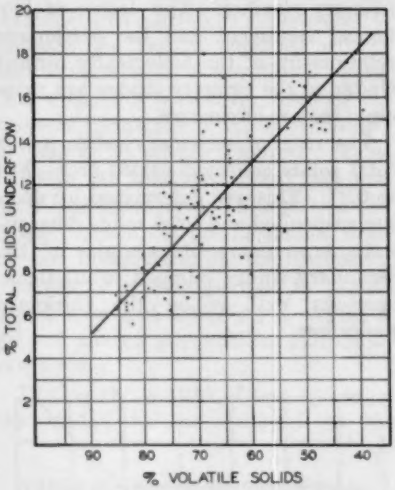

FIGURE 4.—Total solids versus per cent volatile solids for combined primary and secondary sludge.

Concentrated Sludge

The thickened sludge withdrawn from the bottom of the thickener had a "slump" somewhat similar to that of

thick, wet concrete. This sludge (Figure 3) was of exceptional concentration and contained about 15 per cent total solids. The thickened sludge averaged 8.4 per cent total solids for the seven months of operation.

As mentioned previously, the degree to which sewage sludges can be concentrated depends on several variables and their interrelation. One variable already cited is the method of treatment. Different processes cause variance because the flocs formed through various methods of biological treatment are bulky and concentrate to a lesser extent than raw sludge from a primary clarifier. The degree of biological treatment and the percentage relationship of the biologically formed sludge to the primary sludge are principal factors of concern.

The ratio of secondary solids to the total solids collected varied from 0.16 to 0.27. This was determined by making suspended and total solids determinations on composite samples of the secondary sludge pumped to the thickener as well as on the thickener underflow.

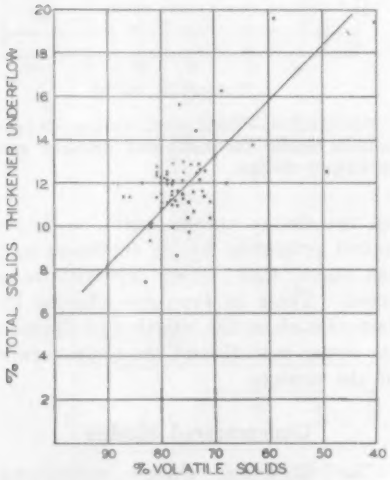

FIGURE 5.—Total solids versus per cent volatile solids for primary sludge only.

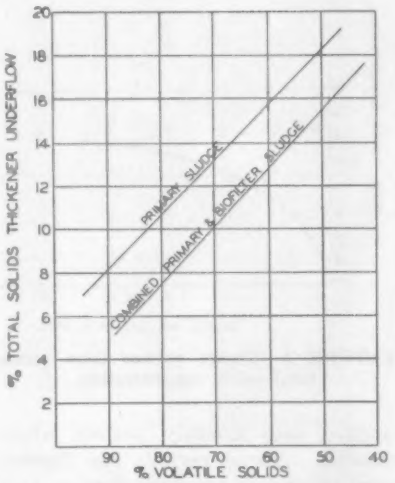

FIGURE 6.—Comparison of increased solids concentration of primary sludge versus combined primary and secondary sludge.

In the concentration of inert substances, the density of the material is important in determining the degree of concentration. The same is true for the concentration of sewage sludge. However, for convenience in analyzing sewage sludge a counterpart measure of the volatile solids content was used. Figure 4 shows a graph of the total solids concentration (primary and secondary sludge) versus the volatile content in the thickened sludge, and indicates that the concentration obtained increases with a decrease in volatile content. Figure 5 shows a similar graph developed from thickening tests on raw sludge only at Ypsilanti, Mich. (5). From a comparison of these two curves (Figure 6) it is evident that primary sludge may be concentrated to a greater extent than combined primary and trickling filter sludge, regardless of volatile content.

Thickener Overflow

Thickener overflow quality commands interest equal to that of the

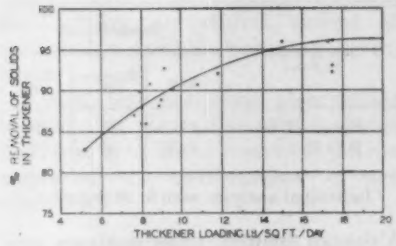

FIGURE 7.—Relationship of solids recovered by thickening versus solids loading.

underflow. Generally the effluent characteristics are similar to or slightly better than those of raw sewage. The 5-day B.O.D. of the overflow approximated 150 p.p.m. The suspended solids varied from 100 p.p.m. to 170 p.p.m. Since the avoidance of anaerobic decomposition is one of the operating criteria, the overflow is entirely suitable for return to the treatment plant. There is no odor problem.

Temperature and sludge volume ratio affect both the sludge concentration and the overflow quality. Another factor is the liquid and solids loading rates. There was little variance of the liquid loading, but the solids loading did vary, and its effect could be observed.

Figure 7 shows that the per cent of solids recovered increased with an increase in solids loading up to a loading of 18 to 20 lb. of solids per square feet per day, when the per cent solids recovered leveled off at approximately 96 per cent. There is a minimum practical solids loading, below which the percentage of solids recovered decreases rapidly. Although percentage-wise the recovery of solids increased, the effluent quality or pounds of solids lost remained constant under the conditions and limits of the test operation.

The effect on the effluent of higher solids loadings cannot be predicted in this case. The maximum solids loading was never accurately determined because the loading to the treatment plant was limited. A solids loading close to 20 lb. per square foot per day was handled during May, 1955, but approximately one-half of this load consisted of well digested solids from the septic tank cleanings. At maximum solids loading the effluent quality begins to either deteriorate or the obtainable solids concentration decreases. These factors in turn are dependent on an interrelation of sludge volume ratio, temperature and the other factors previously mentioned.

Evaluation of Thickener

To evaluate the worth of a sludge thickener installation, it is necessary to compare the results obtained with a unit with those of conventional operation without a thickener. One of the primary comparisons is the difference in volume of sludge pumped. Table II gives the treatment plant flow and daily average gallons of sludge pumped to the digester for the 12 months prior to the installation of the thickener. The sludge volume pumped to the digester averaged 2,150 g.p.d. or 3,900 gal. of sludge per million gallons of raw sewage.

During the months of test operation the sludge volume pumped to the

TABLE II.—Raw Sludge Volume Versus Treatment Plant Flow

Month (1953)	Raw sludge (g.p.d.)	Flow (m.g.d.)
Jan.	2,400	.534
Feb.	2,040	.676
Mar.	2,110	.823
April	2,540	.985
May	2,840	.685
June	2,460	.368
July	1,870	.354
Aug.	2,300	.317
Sept.	2,320	.298
Oct.	1,750	.321
Nov.	1,510	.480
Dec.	1,600	.803
Average	2,150	.533

digester from the thickener averaged only 484 g.p.d. or 1,215 gal. of sludge per million gallons of raw sewage. This is approximately one-third the sludge volume pumped before thickening equipment was installed. A proportionate decrease in digester capacity would appear to be feasible.

In addition to thickening of raw sludge for the digestion process, preliminary tests indicate that thickening of raw sludge prior to vacuum filtration increases production rates and reduces chemical requirements.

The effect of the thickener and its overflow on the entire treatment plant operation is equally as important as reduced sludge volume. Visually, the benefits of thickening resulted in increased clarity of the clarifier effluent, particularly the primary units. This increased clarity can be attributed to two major factors:

1. The digester supernatant load to the primary clarifiers was reduced because of the reduced volume of sludge pumped.
2. Settled sludge was removed immediately from the clarifiers and not allowed to build up.

Although routine daily analyses are not made at the Middleborough treatment plant, periodic samples are taken and analyzed at the Lawrence Experiment Station, Lawrence, Mass. The average results of 12 sets of analyses between March, 1952 and June, 1954 prior to the thickener installation are:

Determination (p.p.m.)	Sewage Sample		
	Raw	Primary Effl.	Final Effl.
Susp. solids	146	50	23
B.O.D.	170	50	13

Three analyses made during the period of thickener operation are:

Determination (p.p.m.)	Sewage Sample		
	Raw	Primary Effl.	Final Effl.
Susp. solids	151	30	28[1]
B.O.D.	145	40	7

[1] Individual analyses were 8, 10 and 67.

Although limited, these analyses confirm the visual observation that there was an improved clarifier performance.

Associated Problems

As with any new process or equipment, the use of sludge thickeners produces a few associated problems that have not been widely encountered before. Primarily, the handling of the thickened sludge presents new pumping and pipe line design considerations due to the increase in the percentage of solids.

An occasional problem is the automatic control of thickened sludge pumping where sewage plant flow varies greatly. This is not a problem for large installations with steady flows. However, some occasional adjustment may be required for a small treatment plant served by a combined sewer system. This was the case at Middleborough when a severe storm would wash additional solids from the streets and catch basins. The sludge blanket in the thickener would build up rapidly due to the suddenly increased solids load. Hence it was necesary to increase the sludge withdrawal temporarily to maintain equilibrium.

Summary and Conclusion

The seven months of full scale thickener operation at Middleborough confirmed that the principles of thickening can be successfully applied to sludges derived from other methods of treatment. During normal operation an average of 3,900 gal. of sludge was pumped to the digester per million gallons of raw sewage compared with an average of only 1,215 gal. of thick-

ened sludge per million gallons of raw sludge during thickening operations.

Visual and analytical observations showed that the inclusion of a sludge thickener tank in the treatment process improved the performance of the clarifiers.

Acknowledgments

The author appreciates the assistance of city officials, Roger McDonald and P. T. Anderson, and sewage treatment plant personnel, F. Sprayberry and T. Churchill, of Middleborough and Engineers L. E. Banks and Paul Weaver.

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CONTRACT AWARDS, THIRD QUARTER, 1955

Contracts for 185 stream pollution abatement projects were awarded by municipalities, during the third quarter of 1955, according to the U. S. Public Health Service. The $61,800,000 represented by these public sewage treatment plant projects covered 98 new treatment facilities and 87 additions, enlargements, or improvements to existing plants. Of the 98 new plants, costing $19,957,000, only 7 were for communities of more than 25,000 population, although these 7 plants accounted for 41.4 per cent of the total new plant cost. These contracts were awarded by: Paducah, Ky.; Las Vegas, Nev.; Middlesex County Sewerage Authority, N. J.; Johnson City, Tenn.; Kingsport, Tenn.; Fort Lewis, Wash.; and King County Sanitary District, Wash.

Municipalities with fewer than 5,000 population awarded 62 contracts for new treatment plants and 22 improvement contracts. Although the number of contracts for the third quarter was substantially below that for the previous quarter (213 contracts), the total dollar value was $10,000,000 more than the value of the second quarter contracts.

Industrial Wastes

BIOLOGICAL TREATMENT OF MIXTURES OF TEXTILE WASTES AND DOMESTIC SEWAGE*

By R. H. Souther and T. A. Alspaugh

Cone Mills Corporation, Greensboro, N. C.

This is the second in a series of papers on pilot plant treatment of highly alkaline textile mill waste and domestic sewage mixtures. The first paper covered the treatment of this waste mixture through a single-stage-high rate trickling filter and activated sludge unit. Results reported in the paper (1) showed that, contrary to published data, a mixture of up to 40 per cent highly alkaline composite dyehouse, bleaching, finishing, mercerizing and color waste can be successfully treated with no chemical pH adjustment. This was found to be practical either through high-rate trickling filter and activated sludge treatment, or through trickling filter treatment alone.

Research in waste problems has been conducted by the textile industry over a long period of time. The outstanding research in textile waste treatment was carried out at one of the Cone mills plants, in the 1930's, by the Textile Foundation (2), jointly sponsored by the State Board of Health, the City of Greensboro, and Cone Mills. In this work it was shown that textile mill waste, including sulfur dye-sewage waste, could be satisfactorily treated by the activated sludge or trickling filter process. The early studies were not carried to the extent of showing that highly alkaline waste could be treated without pretreatment.

* Presented at 1955 Annual Meeting, North Carolina Sewage and Industrial Waste Assn.; Winston-Salem, N. C.; Nov. 14–16, 1955.

Preliminary Investigations

A search of the literature (3) failed to reveal that a 40 to 60 per cent mill waste-sewage mixture had been successfully treated in biological processes at pH up to 11.5. Pretreatment of highly alkaline wastes has been considered a requirement before such waste could be treated biologically with sewage in municipal sewage systems. The city of Charlotte, for example, will not accept waste above a pH of 9.0 in its new biological treatment plant. One objective of research would be to treat without acids or other chemicals, to save expensive chemical costs. If methods can be worked out to reduce alkalinity, such as neutralization by carbon dioxide recovered from power plant stacks the problem of textile waste might be pretty well solved, because neutralized waste is more easily adaptable to biological treatment.

Cone Mills has been actively concerned for a long time with the successful treatment of textile mill waste to relieve stream pollution (4), and has endeavored to determine the effectiveness of biological treatment of highly alkaline textile mill waste-sewage mixtures without the use of expensive chemical pretreatment. It is recognized that research can also benefit by studies to eliminate waste at the source in the plant (5). The application of the Con-Mil redox method (6) to indigo and vat dye evaluations, and sub-

sequent application to sulfur dyeings resulted in further reductions in amount of waste released to the stream by changing the methods of dyeing. Others in the textile industry (7) (8) have made strides in substituting chemicals with low pollution potential for products with high pollution qualities.

This paper reports results obtained in a study of the treatment of textile waste mixture through two stage high rate trickling filters and activated sludge; plus a comparison of the one stage and two stage high-rate filtration and activated sludge results. Biological treatment of domestic sewage-alkaline textile waste mixtures is shown to be practical without use of chemical pretreatment. Waste mixtures with a pH as high as 11.5, a phenolphthalein alkalinity of 562 p.p.m., and a B.O.D. of 558 p.p.m., can be treated through high rate filtration and activated sludge units with a total B.O.D. removal in excess of 90 per cent through all units. The degree of treatment desired can be obtained by increasing or decreasing the size and number of trickling filters, increasing or decreasing loading and/or recirculation on the filters, and by using or omitting the activated sludge units.

Pilot Plant Design

The pilot plant employed in these studies (Figure 1) was designed to give data on several methods of treatment at the same time for comparison purposes. Treatment was provided by the following units:

- Storage and mixing tanks
- Primary settling tank
- First and second stage high-rate trickling filters
- Two large pump sumps for recirculation
- Two small pump sumps for activated sludge feed
- Final settling tank
- Two small laboratory activated sludge units and final settling tanks

The details of design data on various units in this pilot plant can be found in Tables I and II, with the B.O.D. loadings calculated for the various flows investigated. The standard de-

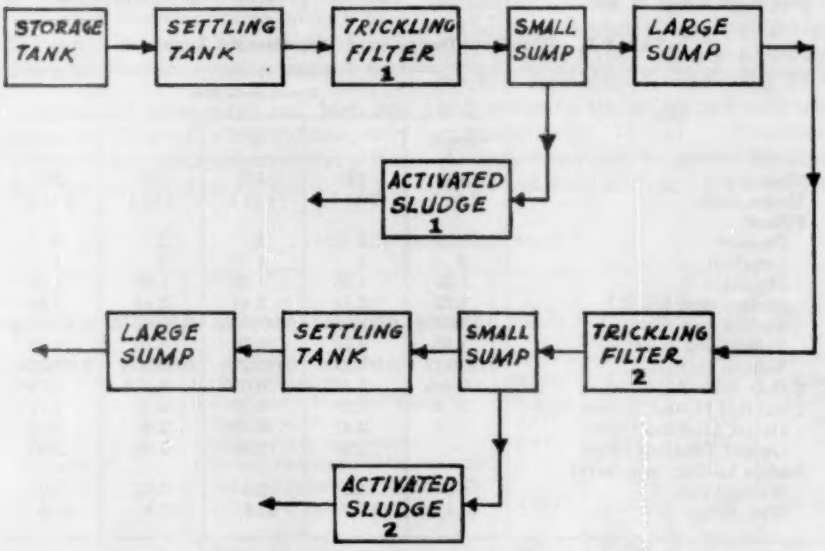

FIGURE 1.—Schematic flow diagram, pilot plant for biological treatment.

TABLE I.—Pilot Plant Design Data—Experimental Units

Primary Settling Tank

Settling time, 2½ hr.
Hoppered bottom with drain for sludge removal.

Final Settling Tank

Settling time, 2½ hr.
Drain in bottom for sludge removal.

Trickling Filters, High-Rate

Depth, 4 ft.
Diameter, 1.25 ft.
Volume loading, variable.
Organic loading, variable.
Stone size:

1st Trickling Filter—Granite Chips

Size (in.)	Pass Screen (%)
1½	0–5
2	0–10
2½	10–20
3	30–60
3½	60–90
4	100

2nd Trickling Filter—Birmingham Slag

Size (in.)	Pass Screen (%)
1½	0–10
2	0–20
3½	85–100
4	100

Activated Sludge Units

Aeration time, 6 hr.
Air quantity. Based on 90 per cent or higher B.O.D. removal on domestic sewage in small pilot plant, 7.5 cu. ft./gal.

sign criteria used for this pilot plant were based on "Suggested Design Data for Sewage Treatment Plants," of the North Carolina State Board of Health.

The pilot plant units were operated continuously 24 hr. a day, seven days a week, except for Saturdays and Sundays when the flow to the plant was reduced and the recirculation rates on the filters increased. The textile mill waste was mixed in the proportions found to exist in a survey of the total waste flow to the North Buffalo Sewage Treatment Plant, Greensboro, N. C. A small percentage of mill waste mixture was first added to the domestic sewage and treated in the pilot plant. Then this percentage was increased until the correct proportion of mill waste to sewage was reached.

In the survey of the total waste picture at the North Buffalo Plant (Table III) it was found that 38.9 per cent of the total waste mixture was mill waste and 61.1 per cent was sewage. It was decided to use a 40 per cent mill waste —60 per cent sewage mixture in this pilot plant study for ease in calculation, and also to use a stronger mixture than the present total combined flow to

TABLE II.—Pilot Plant Design Data—Experimental Runs

Item	Experimental Run				
	Control[1]	1	2	3	4
Flow, g.p.d.	125	125	150	210	250
Recirc. ratio	2 to 1	2 to 1	4 to 1	4 to 1	4 to 1
Filters:					
Number	1	2	2	2	2
Depth, ft.	4	4	4	4	4
Diameter, ft.	1.25	1.25	1.25	1.25	1.25
Surface area (sq. ft.)	1.22	2.44	2.44	2.44	2.44
Surface area (acre)	0.0000278	0.0000556	0.0000556	0.0000556	0.0000556
Volume (cu. ft.)	4.88	9.76	9.76	9.76	9.76
Volume (acre ft.)	0.000112	0.000224	0.000224	0.000224	0.000224
B.O.D. load (lb./day)	0.493	0.473	0.697	0.572	0.790
Total B.O.D. load (lb./cu. yd.)	2.73	1.24	1.73	1.43	1.97
On 1st Trickling Filter	2.73	2.47	3.45	2.86	3.93
On 2nd Trickling Filter	—	1.60	2.51	2.36	3.30
Surface loading (m.g./acre)					
Without recir.	4.5	4.5	5.4	7.55	9.0
With recir.	13.4	13.4	27.0	37.8	45.0

[1] See Ref. 1.

the sewage plant to cover variations in flow and B.O.D. load. A breakdown of the mill waste mixture is shown in Table IV.

Treatment Methods

The three treatment methods, investigated simultaneously for comparison purposes, were as follows:

1. Treatment by primary settling, single stage high-rate trickling filter, activated sludge and final settling.
2. Treatment by primary settling, first stage high-rate trickling filter, second stage high-rate trickling filter and final settling.
3. Treatment by primary settling, first stage high-rate trickling filter, second stage high-rate trickling filter, activated sludge and final settling.

Pilot Plant Experiments

The pilot plant was operated as a one stage high-rate trickling filter—activated sludge plant and as a two stage high-rate trickling filter—activated sludge plant at several different flows in the following order:

125 g.p.d. = recirculation ratio 2 to 1
150 g.p.d. = recirculation ratio 4 to 1
210 g.p.d. = recirculation ratio 4 to 1
250 g.p.d. = recirculation ratio 4 to 1

After the pilot plant had been operated at 125 g.p.d. (recirculation ratio of 2 to 1) and the results tabulated, the flow was increased to 150 g.p.d. It was found that this increase in flow caused the pH in the effluent from the first stage high-rate filter to be too high for successful operation of activated sludge unit No. 1. (Activated sludge unit No. 1 operated on the effluent from the first stage high-rate trickling filter, see Table I.) Therefore, it was necessary to increase the recirculation ratio. The ratio was increased from 2 to 1, to 4 to 1. Increase in the recirculation ratio produced a lower pH in the effluent from the first stage trickling filter, due to the increased dilution factor and also to more opportunity for bacterial action.

Careful calculation of flow-through time indicated that treatment through the first stage high-rate trickling filter and activated sludge units would take approximately 11 hr. Treatment through two-stage high-rate trickling filters and final settling units would be

TABLE III.—Flow to Sewage Plant

Source	B.O.D. (lb./day)	Flow (m.g.d.)	Flow (%)
Sewage	10,230	5.61	—
Unidentified waste[1]	4,670	1.24	—
Total, sewage	14,900	6.85	61.10
Mill waste	20,856	4.36	38.90
Total, all waste	35,756	11.21	100

[1] Not directly traceable to mill outlets so is included under domestic sewage, even though the B.O.D. load is higher than normal domestic sewage.

TABLE IV.—Textile Mill Waste

Plant (no.)	Outlet (no.)	Type of Waste	Flow (m.g.d.)	B.O.D. (lb./day)	B.O.D. (p.p.m.)	Flow (%)
1	1	Sulfur and indigo	0.212	1,820	1,028	4.86
2	2	Rayon and cotton dye waste	0.475	1,510	381	10.90
2	3	Finishing waste	0.518	2,760	640	11.88
3	4	Bleachery and Kier waste	0.144	2,440	2,040	3.30
3 and 4	5	Kier and finishing waste; vat, sulfur, and indigo dye waste; color shop; mercerizing waste	3.011	12,320	490	69.06
		Total	4.36	20,850		100

TABLE V.—Pilot Plant Treatment of 40 Per Cent Highly Alkaline Mill Waste and 60 Per Cent Sewage, No pH Adjustment, Recirculation Ratio 2 to 1 at 125 G.P.D.[1]

Treatment Units	Percent Removal								
Date (Sept., 1954)	1	3	8	10	15	22	24	27	Av.
Primary[2]									
B.O.D.	13.1	7.6	3.6	3.9	2.7	10.8	6.5	0	5.3
Color	0	0	8.3	0	0	0	20.0	0	3.5
First-stage trickling filter[3]									
B.O.D.	32.4	47.5	25.8	33.3	52.7	36.5	28.5	29.5	35.6
Color	40.0	40.0	27.3	29.2	18.2	25.0	12.5	33.3	28.7
Activated sludge No. 1[4]									
B.O.D.	—	—	77.1	57.5	65.2	70.9	40.3	81.9	64.1
Color	—	—	25.0	29.4	38.9	33.3	14.3	25.0	23.6
First-stage trickling filter and activated sludge[2]									
B.O.D.	—	—	83.6	72.8	83.96	83.5	60.1	86.7	78.1
Color	—	—	50.0	50.0	50.0	50.0	40.0	50.0	47.4
Second-stage trickling filter[4]									
B.O.D.	52.1	50.9	69.3	61.6	63.3	72.4	59.7	62.3	62.0
Color	16.7	16.7	12.5	29.4	22.2	33.3	28.6	27.5	24.8
First- and second-stage filtration and final settling[3]									
B.O.D.	74.6	83.1	81.1	75.3	86.5	88.6	73.1	77.8	80.2
Color	54.0	60.0	50.0	53.3	45.5	50.0	60.0	58.3	53.9
Activated sludge No. 2[5]									
B.O.D.	77.2	94.4	93.5	93.98	79.0	82.8	94.3	93.1	88.9
Color	47.8	44.4	64.3	41.7	57.1	50.0	44.0	52.0	51.3
First- and second-stage filtration and activated sludge[2]									
B.O.D.	94.2	97.4	98.6	98.5	96.5	97.3	98.5	98.1	97.4
Color	76.0	72.0	79.2	70.8	72.7	75.0	72.0	80.0	74.8

[1] Load on filters (lb. B.O.D./cu. yd.)
First trickling filter, 2.47.
Second trickling filter, 1.60.
[2] Based on raw influent.
[3] Based on effluent from primary clarifier.
[4] Based on effluent from first-stage trickling filter.
[5] Based on effluent from second-stage trickling filter

7 hr., and treatment through two-stage high-rate trickling filter and activated sludge units would be approximately 12 to 13 hr.

The following method of sampling was used. The waste was collected and pumped into the storage tank at 3 to 4 PM each afternoon. A sample of this new raw mix was collected immediately. At 8 AM the following morning (15 to 17 hr. later) catch samples were collected at all units. The activated sludge units included 6 to 8 hr. storage basins following final settling tanks, and catch samples were collected from these basins. This catch sample method gave enough time for the new waste mixture to go completely through all units of the pilot plant before samples were collected. It was found on sampling, that the raw storage tank waste did not lose biochemical oxygen demand upon standing overnight, but on the contrary it gained about 4 per cent.

Pilot Plant Operation

The waste was mixed daily in a 150-gal. storage tank and was stirred continuously, 24 hr. a day, with a slow speed stirring mechanism to keep suspended matter in suspension. Small sumps served as the feed for the pump to the two activated sludge units. The

waste was pumped 50 ft. from the sumps to either activated sludge unit.

The activated sludge units were necessarily small so that their flow, taken from the total flow would not disrupt the larger trickling filter units. They each used only about one to two gallons per 24 hr. As can be seen, this would make the flow from the first trickling filter to the second trickling filter, and from the second trickling filter to the final settling tank, short by this same amount. To take care of this problem, in the case of the flow of 125 g.p.d., 127 g.p.d. would be pumped from the storage tank each day to the primary settling tank and first trickling filter. Then two gallons would be removed for activated sludge unit No. 1. Then 125 g.p.d. would be pumped to the second filter, and two gallons removed between the filter and final settling tank for activated sludge unit No. 2.

The activated sludge units were first placed in operation using domestic sewage. When the air rate could be adjusted to give 90 to 95 per cent B.O.D. removal continuously, the waste mixture was substituted for sewage and the units were allowed to adjust

TABLE VI.—Pilot Plant Treatment of 40 Per Cent Highly Alkaline Mill Waste and 60 Per Cent Sewage, No pH Adjustment, Recirculation Ratio 4 to 1 at 150 G.P.D.[1]

Treatment Units	Percent Removal								
	October							Nov.	Av.
Date (1954)	6	8	14	15	20	21	29	2	
Primary[2]									
B.O.D.	19.3	8.3	7.05	4.4	15.2	14.8	5.2	8.99	10.5
Color	0	8.3	0	14.3	16.7	0	0	0	5.2
First-stage trickling filter[3]									
B.O.D.	21.3	42.9	35.7	25.9	34.8	7.9	16.9	31.3	27.2
Color	33.3	27.3	42.9	16.7	20.0	20.0	20.0	16.7	25.3
Activated sludge No. 1[4]									
B.O.D.	77.1	82.0	85.8	50.0	79.1	58.2	85.5	75.4	72.95
Color	25.0	25.0	50.0	40.0	0	25.0	25.0	60.0	35.3
First-stage trickling filter and activated sludge[2]									
B.O.D.	85.4	90.6	91.5	64.6	88.5	67.2	88.6	84.6	82.4
Color	50.0	50.0	71.4	57.1	33.3	40.0	60.0	66.7	54.2
Second-stage trickling filter									
B.O.D.	53.6	39.1	45.8	42.8	47.3	36.8	31.97	55.4	43.3
Color	25.0	25.0	25.0	30.0	25.0	25.0	25.0	20.0	24.99
First- and second-stage filtration and final settling[2]									
B.O.D.	71.1	68.9	67.6	59.5	73.6	53.6	48.2	73.9	64.4
Color	50.0	50.0	57.1	50.0	50.0	40.0	40.0	33.3	46.9
Activated sludge No. 2[5]									
B.O.D.	95.8	92.8	88.6	95.2	87.2	94.9	92.5	88.97	92.5
Color	50.0	25.0	40.0	48.6	58.3	50.0	50.0	62.5	48.6
First- and second-stage filtration and activated sludge[2]									
B.O.D.	98.8	97.8	96.3	98.1	96.3	97.5	96.0	96.9	97.2
Color	75.0	62.5	74.3	74.3	79.2	70.0	70.0	75.0	72.7

[1] Load on filters (lb. B.O.D./cu. yd.)
First trickling filter, 3.45.
Second trickling filter, 2.51.
[2] Based on raw influent.
[3] Based on effluent from primary clarifier.
[4] Based on effluent from first-stage trickling filter.
[5] Based on effluent from second-stage trickling filter.

TABLE VII.—Pilot Plant Treatment of 40 Per Cent Highly Alkaline Mill Waste and 60 Per Cent Sewage, No pH Adjustment, Recirculation Ratio 4 to 1 at 210 G.P.D.[1]

Treatment Units	Percent Removal									
Date (Mar., 1955)	9	10	11	16	17	18	23	24	25	Av.
Primary[2]										
B.O.D.	22.6	0	2.98	14.1	5.8	16.5	9.1	5.1	6.4	9.2
Color	0	0	0	0	0	0	0	0	0	0
First-stage trickling filter[3]										
B.O.D.	0	30.5	6.8	13.7	24.2	21.97	0	30.4	15.7	17.6
Color	28.6	25.0	16.7	25.0	25.0	25.0	33.3	25.0	25.0	25.4
Activated sludge No. 1[4]										
B.O.D.	64.2	65.1	84.9	67.7	64.7	76.5	51.6	55.3	61.5	66.7
Color	40.0	33.3	30.0	33.3	46.6	60.0	25.0	55.5	55.5	42.3
First-stage trickling filter and activated sludge[2]										
B.O.D.	72.3	75.7	86.4	76.2	74.8	85.4	55.9	70.5	69.1	75.1
Color	57.1	50.0	41.6	50.0	60.0	70.0	50.0	66.6	66.6	56.9
Second-stage trickling filter[4]										
B.O.D.	47.6	47.4	52.7	31.9	39.8	53.3	41.3	15.5	38.1	40.1
Color	20.0	22.2	20.0	22.2	27.7	22.2	25.0	16.6	22.2	22.1
First- and second-stage filtration and final settling[2]										
B.O.D.	58.8	64.9	59.9	49.5	59.3	71.0	46.6	49.2	51.1	57.0
Color	42.8	41.6	33.3	41.6	45.8	41.6	50.0	37.5	41.6	41.8
Activated sludge No. 2[5]										
B.O.D.	91.6	81.1	89.7	77.5	73.3	86.6	73.9	87.1	67.3	80.7
Color	60.0	54.2	62.5	54.2	44.6	60.0	40.0	57.3	54.2	54.6
First- and second-stage filtration and activated sludge[2]										
B.O.D.	96.3	92.6	95.6	88.6	88.5	96.1	86.1	92.8	84.0	91.3
Color	97.1	73.3	88.6	73.3	70.0	76.6	70.0	73.3	73.3	73.6

[1] Load on filters (lb. B.O.D./cu. yd.)
First trickling filter, 2.86.
Second trickling filter, 2.36.
[2] Based on raw influent.
[3] Based on effluent from primary clarifier.
[4] Based on effluent from first-stage trickling filter.
[5] Based on effluent from second-stage trickling filter.

themselves. It was found on this waste mixture that if the suspended solids content of the activated sludge units was maintained higher than the normal practice, better results would be obtained. The suspended solids content was held as close to 7,000 to 9,000 p.p.m. as possible, instead of 2,000 to 3,000 p.p.m. This was the range of best operation. Lower suspended solids levels caused foaming, but when the level was raised foaming stopped.

When the flow reached 250 g.p.d., the volume loading was so high that all growth on the first 4 to 6 in. of stone in the filters washed away. The sloughing of slime growth from the filters was also very serious, much more than at lower rates.

Pilot Plant Results

The results of biological treatment of highly alkaline textile mill waste and domestic sewage mixtures (with no chemical pretreatment), are summarized in several tables.

Table V presents data on the B.O.D. and color removal accomplished by all combinations of units, while operating the pilot units at a flow rate of 125 g.p.d. (recirculation ratio 2 to 1).

Table VI presents the same data at a flow rate of 150 g.p.d. (recirculation ratio 4 to 1). The data in Tables VII

TABLE VIII.—Pilot Plant Treatment of 40 Per Cent Highly Alkaline Mill Waste and 60 Per Cent Domestic Sewage, No pH Adjustment, Recirculation Ratio 4 to 1 at 250 G.P.D.[1]

Treatment Units	Percent Removal													
	March		April										May	Av.
Date (1955)	30	31	1	6	14	15	20	21	22	27	28	29	4	
Primary[2]														
B.O.D.	4.45	15.2	2.35	0	0	22.7	27.0	26.7	11.4	3.8	0	0	5.4	9.3
Color	0	0	0	0	0	0	0	0	0	0	0	0	0	0
First-stage trickling filter[3]														
B.O.D.	24.8	6.0	0	4.74	17.4	33.3	22.5	19.5	5.4	20.3	21.5	20.0	15.0	16.3
Color	25.0	25.0	25.0	0	23.0	28.6	16.6	11.1	10.0	11.1	11.1	14.3	14.3	15.6
Activated sludge No. 1[4]														
B.O.D.	58.8	70.4	46.7	57.9	67.2	66.9	54.4	37.2	56.2	55.6	57.5	16.5	63.0	54.0
Color	22.2	33.3	22.2	42.9	30.0	30.0	30.0	43.9	44.4	44.4	44.4	33.3	25.0	35.7
First-stage trickling filter and activated sludge[2]														
B.O.D.	69.9	76.0	48.0	59.0	72.9	82.8	74.2	63.0	64.0	66.2	66.5	33.2	70.3	66.0
Color	41.6	50.0	41.6	42.7	46.2	50.0	41.6	50.0	50.0	50.0	50.0	42.8	35.6	46.0
Second-stage trickling filter[4]														
B.O.D.	43.2	25.8	30.9	24.7	20.4	28.9	27.4	12.7	26.2	15.8	31.3	34.5	30.6	27.3
Color	22.2	11.1	11.1	14.3	20.0	20.0	20.0	25.0	11.1	12.5	12.5	33.3	33.3	18.7
First- and second-stage filtration and final settling[2]														
B.O.D.	59.2	44.2	32.6	31.2	45.7	69.1	55.1	52.4	42.0	36.2	46.0	55.5	51.9	48.0
Color	41.7	33.3	33.3	14.3	38.5	42.9	33.3	33.3	20.0	22.2	22.2	42.9	42.9	31.5
Activated sludge No. 2[5]														
B.O.D.	54.5	87.8	77.5	74.5	63.0	65.1	60.0	73.3	65.4	55.9	74.5	49.6	60.3	67.5
Color	17.1	57.5	57.5	63.3	37.5	57.5	37.5	50.0	50.0	50.0	50.0	25.0	12.5	45.0
First- and second-stage filtration and activated sludge[3]														
B.O.D.	81.4	92.8	85.0	81.9	75.6	87.0	83.5	86.0	79.0	71.3	86.0	73.7	79.5	82.0
Color	51.5	71.8	71.8	68.8	61.5	75.6	64.2	66.7	60.0	61.0	61.0	57.2	50.0	62.3

[1] Load on filters (lb. B.O.D./cu. yd.)
First trickling filter, 3.93.
Second trickling filter, 3.30.
[2] Based on raw influent.
[3] Based on effluent from primary clarifier.
[4] Based on effluent from first-stage trickling filter.
[5] Based on effluent from second-stage trickling filter.

and VIII are based on flow rates of 210 and 250 g.p.d., respectively (both using recirculation ratios of 4 to1).

Table IX presents a comparison of pH reduction through the various units of the pilot treatment plant.

Table X presents data on the removal of suspended solids accomplished by the combination of treatment units at the several flow rates employed.

Discussion

These pilot plant studies have shown that mixtures of textile mill waste and domestic sewage can be successfully treated together by biological processes.

The high pH of a waste affects the per cent removal of B.O.D by a trickling filter. In these experiments it was noted that in all flows except at 250 g.p.d., the filters reduced the pH of the waste flowing through them, at least sufficiently for the activated sludge units to take up the extra B.O.D. load which was due to the reduced efficiencies of these filters.

At 250 g.p.d., the rate of flow on the filters was so high (45 m.g. per acre, 80 per cent above standard design practice upper limit) that the filter growth was washed through. All the growth

TABLE IX.—Comparison of the pH Reduction Through Various Units

Treatment Units	Monthly Average pH			
Flow rate (g.p.d.)	125	150	210	250
Pilot plant infl.	10.95	11.49	10.8	10.7
Primary effl.	10.86	11.46	10.8	10.7
First trickling filter effl.	9.75	10.5	10.1	10.1
Second trickling filter effl.	9.01	9.75	9.6	9.6
Final settling tank effl.	9.0	9.74	9.5	9.58
Activated sludge No. 1 effl.[1]	8.77	9.29	9.0	9.3
Activated sludge No. 2 effl.[2]	8.4	8.74	8.5	8.9

[1] From first trickling filter.
[2] From second trickling filter.

TABLE X.—Percent Removal of Suspended Solids through the Pilot Plant

Date	Flow (g.p.d.)	Suspended Solids Removal, Per cent								
		Primary	First Trickling Filter Only	Second Trickling Filter Only	Raw through Second Trickling Filter	Raw through Final Settling	Activated Sludge No. 1 Only	Activated Sludge No. 2 Only	Raw through Activated Sludge No. 1 (1 Trick. Filter and Act. Sl.)	Raw through Activated Sludge No. 2 (2 Trick. Filters and Act. Sl.)
1954										
Sept. 10	125	0	50	12	56	58	80	88.5	90	95
Oct. 8	150	0	47.3	13.7	54.4	56.7	45.2	75.5	71	89
1955										
Mar. 9	210	0	34.4	15.8	44.9	44.9	53.5	87.5	69.8	90.8
April 6	250	0	30.0	30.5	48.8	44.6	28.5	70.5	52.2	85

was washed from the first six inches of stone in both filters. The effluent pH from the second-stage filter to the activated sludge unit was above 9.5, and as a result the efficiency of the activated sludge unit dropped.

Roughing filters will lower the pH, reduce the B.O.D. load, and will change the alkalinity from hydroxides to carbonates and bicarbonates. This will allow a constant pH and B.O.D. load to be fed to the activated sludge units from the filters and the waste can be very satisfactorily treated. There is a limit, though, to the volume and organic loading roughing filters will assimilate. These limits should be taken into consideration, especially the volume or surface loading limit.

Even at a pH of 11.5 and B.O.D. of 558 p.p.m., if enough filter capacity is provided the waste can be conditioned successfully for secondary treatment by activated sludge with a total B.O.D. removal in excess of 90 per cent through all units.

The filters and activated sludge must be successfully acclimated gradually to this highly alkaline waste mixture for good treatment.

In treating the 40 per cent to 60 per cent mill waste—sewage good color removal was accomplished even at a pH of 11.5 at normal B.O.D. loading; 70 per cent removal through the whole process. (The effluent was clear, with only a faint yellow tinge, and not offensive.) The color removal dropped to 50 per cent at high volume loading.

The final effluent had no odor, and no strong sulfide odors were noticed at any of the units. The fact that the treatment was carried out in a highly alkaline state allowed the sulfides to be converted to sulfates instead of coming off into the atmosphere as hydrogen sulfide.

The degree of suspended solids removal was excellent. The effluent was clear with no turbidity to be seen. This was accomplished at flow rates ranging from 125 g.p.d. to 210 g.p.d.

TABLE XI.—Summary Comparison

One Stage Filtration Followed by Activated Sludge

Flow (g.p.d.)	Recirculation Ratio	pH Value	B.O.D. (p.p.m.)		B.O.D. Removal (%)
			Influent	Effluent	
125	2 to 1	10.95	455	99.5	78
150	4 to 1	11.49	558	98.4	82.4
210	4 to 1	10.8	326	81.3	75.1
250	4 to 1	10.7	379	128	66.0

TABLE XII.—Summary Comparison
Two Stage Filtration Followed by Settling

Flow (g.p.d.)	Recirculation Ratio	pH Value	B.O.D. (p.p.m.)		B.O.D. Removal (%)
			Influent	Effluent	
125	2 to 1	10.95	455	90	80
150	4 to 1	11.49	558	199	64.4
210	4 to 1	10.8	326	140	57.0
250	4 to 1	10.7	379	197	48.0

At flow of 250 g.p.d., the removal dropped from 95 per cent to 85 per cent, due to higher volume loading.

Sulfides and sulfites were converted to sulfates, as evidenced by a rise in sulfate values through the pilot plant and reduction of the sulfites and sulfides.

The activated sludge units seem to operate best with a suspended solids content of from 7,000 to 9,000 p.p.m., instead of the normal 2,000 to 3,000 p.p.m. No foaming was evident, and the sludge index was usually below 100, except when the pH value increased sharply in the units. Operating at pH's in the range of 9.5 to 10.0, there was no bulking at the lower flows and B.O.D. loadings. The sludge index increased with higher flows and B.O.D. loadings and bulking occurred in the pH range of 9.8 to 10.5.

These experiments proved that trickling filters will serve to reduce the pH and alkalinity of a waste, thus making the waste treatable by second-stage treatment-activated sludge. The filters do not remove a large amount of the B.O.D. load but do start biological action.

Summary Comparison of One-Stage Filtration and Activated Sludge

The results of treatment through one-stage high-rate filtration and activated sludge units produces an effluent which may be satisfactory for direct disposal to a small water-way. Table XI summarizes B.O.D. in the original waste, per cent removal of B.O.D., and p.p.m. of B.O.D. remaining in the effluent.

Summary Comparison of Two-Stage Filtration and Final Settling

The results of treatment through two-stage filtration and final settling do not give as good an effluent as treatment through one-stage filtration and activated sludge. This was as expected. There was one exception, at a flow of 125 g.p.d. and recirculation ratio of 2 to 1. (See Table XII.)

Summary Comparison of Two-Stage Filtration and Activated Sludge

The results of treatment through two-stage filtration and an activated

TABLE XIII.—Summary Comparison
Two Stage Filtration Followed by Activated Sludge

Flow (g.p.d.)	Recirculation Ratio	pH Value	B.O.D. (p.p.m.)		B.O.D. Removal (%)
			Influent	Effluent	
125	2 to 1	10.95	455	12	97.4
150	4 to 1	11.49	558	16	97.2
210	4 to 1	10.8	326	28	91.3
250	4 to 1	10.7	379	68	81.5

sludge unit were excellent. (See Table XIII.) The two-stage filtration units will give a better degree of treatment within certain limits using the same amount of stone as for one stage, but initial cost and maintenance would be higher.

Future Experiments

Experiments are in progress to compare combined biological treatment of mill waste-sewage mixtures without acid, and separate treatment of mill waste alone with acid pretreatment.

Acknowledgment

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EFFECTS OF HIGH CONCENTRATIONS OF NITROGEN ON ACTIVATED SLUDGE

By Robert L. Taylor, Everett R. Mathews, and C. W. Christenson

Respectively, Chemist AEC Materials and Accountability Branch, San Francisco Operations Office, Oakland, Calif.; Chief, Health Engineering Branch, Santa Fe Operations Office; and Group Leader, H–7, Los Alamos Scientific Laboratory, Los Alamos, N. Mex.

The cause of occasional malfunction of the activated sludge process in treatment of a liquid industrial waste was thought to be caused by high concentrations of either nitrate ion or ammonium ion which occurred in the raw waste at irregular intervals. To investigate each of these possibilities, experimental activated sludges were established in laboratory units, and the effects of various concentrations of nitrate and ammonium nitrogen were studied. The usual chemical control tests were made to assist in interpreting results from these studies.

Experimental Results

Two experimental activated sludges were set up by placing equal amounts of sewage-fed stock sludge in each of two 10-l. calibrated, Pyrex bottles. These sludges were aerated continuously by applying compressed air through diffuser stones. Each sludge was fed settled sewage, once daily for six days each week, by the fill and draw method; each drawdown reduced the volume to 3 l. and each feed volume was 5 l. The first of the two sludges served as a control, while the second was the experimental sludge. To the experimental sludge was added each day, in addition to the feed of settled sewage, supplemental nitrate ion in the form of sodium nitrate. The sludge effluents were analyzed twice each week and the mixed liquor at weekly intervals.

Nitrate Nitrogen Addition

After some exploratory changes in procedure, the daily nitrate supplement to the experimental sludge was raised by increments of 160 p.p.m. nitrate nitrogen, based on the feed volume of 5 l., to 1,120 p.p.m. A period of two weeks to one month was allowed at each supplementation level for acclimation. When the 1,120 p.p.m. level of nitrate supplementation was reached, control of the mixed liquor suspended solids content was begun. After the weekly analysis for suspended solids was completed, the fraction of mixed liquor calculated to reduce the suspended solids to 3,000 p.p.m. was wasted prior to drawdown; if the analysis showed less than 3,000 p.p.m. suspended solids, no sludge was wasted.

Finally, in an attempt to reach a definitive point in an experiment tending to become of inconvenient duration, the nitrate supplementation level was successively increased from 1,120 p.p.m. to 1,600, 2,400, 3,600 and 4,800 p.p.m. during the latter part of the study. No adverse effects were noted as a result of these more abrupt changes.

A summary of the data obtained during the course of the experiment is given in Table I. Inspection of these data shows that the only significant differences between the experimental and control sludges are in sludge density indices and, of course, the experimentally induced differences in effluent nitrate contents. The experimental

TABLE I.—Summary of Data on Effect of Nitrate Nitrogen on Sewage-Fed Activated Sludges

Determination (p.p.m.)	Period (days)										
	a	b	c	d	e	f	g	h	i	j	k
	15	10	12	37	35	22	28	17	15	8	7
Raw Feed											
NO_3—N to exp'l sl.	40	368	460	640	800	960	1,120	1,600	2,400	3,600	4,800
Effluent											
NO_3—N in control	49	41	32	40	21	21	25	—	56	51	45
NO_3—N in exp'l sl.	74	373	527	793	920	988	1,127	1,712	2,480	2,822	4,433
NO_2—N in control	—	—	—	—	—	—	—	—	0	0	0
NO_2—N in exp'l sl.	—	—	—	—	—	—	—	2.4	0	0	0
NH_4—N in control	0.8	0	0	1.2	0.6	0.3	1.5	—	0	0	0
NH_4—N in exp'l sl.	0.8	0	0	1.0	0.8	0.4	1.5	0.3	0	0	0
O.C. in control[1]	19	2	16	29	40	38	42	—	46	28	39
O.C. in exp'l sl.[1]	15	4	20	31	46	45	37	38	19	4	65
B.O.D. in control	—	—	—	—	2	3	7	—	12	11	4
B.O.D. in exp'l sl.	—	—	—	—	3	2	6	6	2	3	9
PO_4 in control	19	16	20	23	25	29	27	—	29	28	28
PO_4 in exp'l sl.	17	16	19	21	23	25	26	—	30	23	30
pH in control[2]	6.5	7.3	7.5	6.8	7.8	7.7	7.6	—	6.9	6.9	6.7
pH in exp'l sl.[2]	6.4	7.4	7.5	7.0	7.9	7.8	7.7	7.5	7.3	7.2	7.1
Mixed Liquor											
Susp. sol. in control	2,391	2,597	2,674	2,481	2,636	3,267	3,215	—	2,396	2,174	2,222
Susp. sol. in exp'l sl.	2,285	2,671	2,854	2,603	3,079	3,666	2,987	3,342	2,953	3,190	3,170
Sludge density index in control	0.62	0.61	0.30	—	0.71	0.60	0.75	—	0.93	0.78	0.79
Sludge density index in exp'l sl.	0.62	0.72	0.76	—	1.86	2.31	3.07	2.40	1.60	1.77	1.27

[1] Acid dichromate oxygen consumed procedure.
[2] Median value.

sludge rapidly became more dense than the control sludge and remained this way. The density of the experimental sludge was such that the sludge acquired a marked coarseness, or granularity, to an extent that moderately increased agitation was necessary to maintain the sludge in suspension in the mixed liquor. However, none of the various analyses conducted indicated that operation of this coarse sludge was unsatisfactory. Whether the coarseness of the experimental sludge was caused by a specific action by the nitrate ion, or was the result of the high electrolyte concentration in the mixed liquor, has not been investigated.

Sawyer and Bradney (1) have reported difficulty with rising sludge in settling tanks when the nitrate nitrogen content exceeded 10 p.p.m. No such difficulty was encountered with these experimental sludges nor in those described in the following section on ammonium nitrogen supplementation. Presumably the difference in operating experience is due to the relatively long aeration period (about 22 hr.) and short settling time (1 hr. or less) used for this experimental work.

When considering the suspended solids values for the periods "h" through "k" (Table I), a direct comparison between the control and experimental sludges should not be made, since it was necessary to develop a new control prior to period "h"; the original control sludge was accidentally lost and suspended solids content of the new control had not reached 3,000 p.p.m. when control analyses were resumed. However, the data that are presented and observations made during the course of the experiment indicate that a relatively high concentration of nitrate nitrogen ion does not inhibit sludge growth or the normal metabolic processes connected with such growth.

Nitrate Content Variations

The nitrate nitrogen analytical values reported in Table I show the somewhat unsatisfactory result of attempts to compare nitrate added to the raw feed and nitrate content of the effluents as determined by the phenol-

disulfonic acid colorimetric method. Occasional extreme variations in the values obtained by the day-to-day analyses were encountered, and utilization of other methods of analysis did not clarify the situation.

Many postulates have been discussed as possible explanations for these variations, such as nitrate utilization by the sludge, adsorption of nitrate by the sludge, presence in the sewage feed of substances which interfere with the analytical procedure, and accidental omission of the daily nitrate supplement. Any or all of these postulates are possible, but it is believed the least probable are those which assume sudden changes in the sludge metabolic processes. Further, the variations were sufficiently frequent to reduce markedly the probability that the error was accidental omission of the nitrate supplement. The average effluent nitrate nitrogen analytical values for each level of nitrate supplementation, both with and without the highly suspected values, are sometimes higher and sometimes lower than the theoretical values. The source of the apparent variations, therefore, would seem to lie in the analytical procedure, and to be caused by the presence of substances which interfere with the color development.

Infrequent foaming, with slight sludge loss, was observed in the experimental sludge; and infrequent production by both sludges of clear brown effluents. These variations are attributed to the varying composition of the sewage feed. No correlation was observed between either of these phenomena and variation of effluent nitrate content.

TABLE II.—Summary of Data on Effect of Ammonium Nitrogen on Sewage-Fed Activated Sludges

Determination (p.p.m.)	Period (days)					
	a	b	c	d	e	f
	19	24	12	19	15	28
Raw Feed:						
NH_4—N to exp'l sl.	144	160	176	208	320	480
B.O.D. to N ratio in exp'l sl.	0.8	0.8	0.7	0.7	0.3	0.2
Effluent:						
NH_4—N in control	0	0	—	0.5	0	0
NH_4—N in exp'l sl.	1.4	3	2.4	5.1	13.7	211
NO_2—N in control	0	0	—	5.9	0	0
NO_2—N in exp'l sl.	0	0	0	1.9	0	220
NO_3—N in control	32	33	—	6	45	43
NO_3—N in exp'l sl.	129	139	160	260	340	100
O.C. in control[1]	37	45	—	68	43	24
O.C. in exp'l sl.[1]	47	50	64	52	35	249
B.O.D. in control	4	3	—	15	12	7
B.O.D. in exp'l sl.	3	7	9	7	4	26
pH in control[2]	6.8	7.2	—	7.4	7.0	7.0
pH in exp'l sl.[2]	6.4	6.8	6.6	7.1	6.4	6.9
PO_4 in control	37	30	—	35	28	25
PO_4 in exp'l sl.	28	27	41	69	23	12
Mixed Liquor:						
Susp. sol. in control	2,681	2,774	—	3,128	2,396	2,222
Susp. sol. in exp'l sl.	3,027	2,874	2,699	3,016	2,908	2,669
Sludge density index in control	0.59	0.72	—	0.90	0.92	0.79
Sludge density index in exp'l sl.	1.69	1.79	1.86	1.71	1.72	1.99

[1] Acid dichromate oxygen consumed procedure.
[2] Median value.

Ammonium Nitrogen Addition

To extend the studies initiated with the nitrate ion investigation, two more experimental activated sludges were established from sewage-fed stock, and to one of these various concentrations of ammonium nitrogen were added. The same volumes described for the nitrate study, as well as the same drawdown and feeding procedures, were used. To the feed of 5 l. of settled sewage ammonium ion was added in the form of various salts, in increments shown in Table II, with time provided for acclimation at each level of supplementation.

The ammonium ion was added as various salts rather than as the same salt throughout the course of the experiment for several reasons. The original salt, ammonium sulfate, was chosen for stability and lack of effect of the anion. However, when low terminal pH values developed with this supplement, a change was made to ammonium bicarbonate; the latter was selected with the possibility in mind that the bicarbonate ion might serve to neutralize some of the acid produced metabolically, thus keeping the terminal pH values in a more desirable range.

Neutralization by bicarbonate ion was only temporarily effective, and it soon became necessary to add caustic soda routinely to the mixed liquor, in addition to the feed and ammonium ion supplement. Later the supply of ammonium bicarbonate was exhausted and ammonium carbonate was substituted with no noticeable differences ensuing; addition of caustic soda was still necessary to control the terminal pH. The possible loss of free ammonia from the mixed liquor by aeration, during the somewhat more alkaline period following addition of caustic soda, was investigated and shown to be negligible.

Inspection of the data in Table II shows that the values for dichromate oxygen consumed, ammonium nitrogen, nitrite nitrogen and nitrate nitrogen are most indicative of the effects of the ammonium nitrogen supplement. These parameters for the experimental sludge are plotted in Figure 1. They show that marked changes occurred in the performance of this sludge by the time an ammonium nitrogen concentration in the raw feed of 480 p.p.m. was reached. The oxygen consumed, B.O.D., ammonium nitrogen and nitrite nitrogen values in the effluent increased markedly, while the nitrate nitrogen values decreased, when ammonium nitrogen supplementation was increased from 320 to 480 p.p.m.; the corresponding B.O.D. to nitrogen ratios in the raw feed were 0.3 to 1 and 0.2 to 1. These changes, of course, resulted in undesirable effluent characteristics.

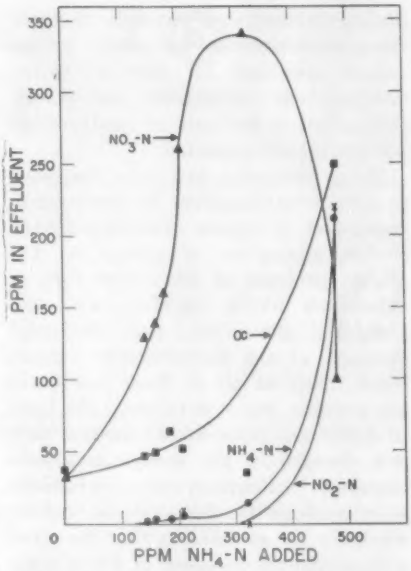

FIGURE 1.—Effect of ammonium nitrogen on sewage-fed activated sludge.

Synthetic Feed Studied

In order to obtain more information on B.O.D. to nitrogen ratio in the raw feed, and at the same time to eliminate the variations inherent in the sewage feed, a control sludge and an experi-

mental sludge were established on a synthetic feed of white flour, sugar, minerals and tap water.

Once a day, six days each week, the 8-l. aeration volumes were drawn down to 3 l., and the 5-l. effluent volumes were replaced with tap water. Slurries of a flour-sugar mixture and a mineral supplement were then added to produce concentrations of 500 p.p.m. flour, 300 p.p.m. sugar, 4.8 p.p.m. magnesium, 14.7 p.p.m. phosphate and 3.2 p.p.m. potassium, based on the 5-l. feed volume, in addition to the minerals present in the Los Alamos tap water. As with the sewage-fed experimental sludge, various salts of ammonium ion were added to the synthetic-fed experimental sludge, in increments shown in Table III, with time for acclimation to each new level.

Examination of the data in Table III shows that at an ammonium nitrogen feed concentration of 112 p.p.m., the effluents from the flour-sugar-fed activated sludge began to indicate undesirable characteristics; as the supplementary ammonium nitrogen dosage was increased these effluents departed still further from generally accepted standards. At this concentration of ammonium nitrogen in the feed, the B.O.D. to nitrogen ratio was 3.2 to 1.

Increasing the B.O.D. to nitrogen ratio in the feed to more normal levels, by maintaining the ammonium ion supplement constant at the highest level reached and adding a high-carbon food supplement, might have been investigated to determine whether or not the sludges would re-establish satisfactory effluent characteristics. This further step was not taken because the experiment already had consumed far more time than had originally been allotted for it.

TABLE III.—Summary of Data on Effect of Ammonium Nitrogen on Flour-Sugar-Fed Activated Sludges

Determination (p.p.m.)	Period (days)								
	a	b	c	d	e	f	g	h	i
	13	19	9	14	12	19	15	13	14
Raw Feed:									
NH_4—N to exp'l sl.	40	56	72	88	112	136	216	320	480
B.O.D. to N ratio in expl'l sl.	7.7	5.8	4.7	3.9	3.2	2.6	1.7	1.2	0.8
Effluent:									
NH_4—N in control	0	0.1	0.1	0	0	0.2	0	0	1.0
NH_4—N in exp'l sl.	0.1	0.8	4	0	0	0	27	69	137
NO_2—N in control	0	0	0	0	0	0	0	0.2	0.6
NO_2—N in exp'l sl.	0	0.1	0.5	0.2	15	32	39	146	203
NO_3—N in control	0.4	0.3	0.4	0.4	0.2	1.3	0.3	0.5	1.0
NO_3—N in exp'l sl.	14	30	36	66	79	77	62	17	5
O.C. in control[1]	16	13	22	23	20	50	20	69	83
O.C. in exp'l sl.[1]	35	19	20	14	36	69	118	211	319
B.O.D. in control	3	3	4	7	8	21	14	57	26
B.O.D. in exp'l sl.	18	6	9	5	5	20	35	47	54
pH in control[2]	7.9	7.9	7.8	7.8	7.7	7.7	7.6	7.9	8.0
pH in exp'l sl.[2]	7.2	7.0	8.0	7.8	7.6	7.6	7.4	7.1	6.5
PO_4 in control	5	7	11	6	4	16	3	6	6
PO_4 in exp'l sl.	3	0.7	0.5	0	0	0.8	5	8	5
Mixed Liquor:									
Susp. sol. in control	4,705	4,256	4,029	4,212	4,180	3,925	3,448	4,375	3,782
Susp. sol. in exp'l sl.	3,996	3,906	3,614	3,797	3,688	4,473	3,072	3,642	3,307
Sludge density index in control	3.49	3.86	2.59	3.11	3.99	2.56	2.03	1.81	—
Sludge density index in exp'l sl.	2.08	2.57	1.85	1.22	0.73	1.37	1.81	1.67	—

[1] Acid dichromate oxygen consumed procedure.
[2] Median value.

Discussion of Results

High concentrations of nitrate nitrogen, added to an experimental activated sludge over periods of time permitting acclimation, did not inhibit sludge growth or the normal metabolic processes. The only effect noted was an increase in the density of the sludge. Therefore, it is probable that high nitrate feed concentrations were not the cause for the occasional deterioration of effluents at the local industrial activated sludge treatment plant.

High concentrations of ammonium nitrogen caused deterioration of activated sludge effluents. The effect was noted at a B.O.D. to nitrogen ratio of 3.2 to 1 for the flour-sugar-fed sludge and a ratio of 0.2 to 1 for the sewage-fed sludges. These can be compared to an optimum ratio of 20 to 1 as suggested by Heukelekian (2). At these distorted ratios, the amount of B.O.D. present appeared inadequate to support the biological growth necessary for oxidizing the ammonium nitrogen to nitrate. The fact that the sewage-fed sludge was able to endure a greater distortion in the feed ratio can be attributed to the generally greater resistance to any stress of such sludges over those fed only synthetic materials. It is believed that deterioration of the effluents was due to a deficiency of B.O.D. rather than to any specific toxic effect of the ammonium ion. From these experiments it is reasonable to infer that high concentrations of ammonium nitrogen could have been the cause of occasional malfunction of the local industrial activated sludge treatment plant.

Conclusions

Judged by acceptable effluent composition and continued sludge growth, nitrate nitrogen has no deleterious effect on activated sludge in concentrations up to 4,800 p.p.m., if an adequate period is allowed for acclimation.

Undesirable activated sludge effluent characteristics appear when high concentrations of ammonium nitrogen are present in the raw feed, even though adequate time is allowed for acclimation. With sewage-fed sludge these characteristics appeared at a concentration of 480 p.p.m., and a B.O.D. to nitrogen ratio of 0.2 to 1. With a flour-sugar-mineral-fed sludge, these characteristics appeared at an ammonium nitrogen concentration of 112 p.p.m., and a B.O.D. to nitrogen ratio of 3.2 to 1.

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HIGH-RATE ACTIVATED SLUDGE TREATMENT OF FINE CHEMICAL WASTES *

By F. E. Dryden, P. A. Barrett, J. C. Kissinger, and W. W. Eckenfelder, Jr.

Respectively, Waste Treatment Department Head, Associate Research Chemical Engineer and Waste Treatment Department Supervisor, Chemical Division, Merck & Co., Inc., Danville, Pa.; and Consulting Engineer, Weston, Eckenfelder and Hood, Paterson, N. J.

The Cherokee Plant of Merck & Co., Inc., is a medium sized medicinal and fine chemical manufacturing plant located on the North Branch of the Susquehanna River near Danville, Pa.

The principal products manufactured are antibiotics produced by fermentation, synthetic vitamins and cortisone. The wastes from these processes constitute the disposal problem.

Before manufacturing operations were started in the summer of 1951, a waste discharge permit was obtained from the Pennsylvania Department of Health which stipulated that a 35 per cent reduction in B. O. D. must be attained, essentially all the suspended solids must be removed, and the amount of B.O.D. discharged in the treatment plant effluent must not exceed a prescribed limit.

The chemical treatment facilities provided to meet the conditions of this permit consisted of neutralization, equalization, alum treatment and the necessary sludge handling facilities. Treatment and recovery of certain waste products within the manufacturing facilities also were planned. The basis of design for this waste treatment installation was the calculated B.O.D. load of the wastes plus consideration of the experience at the Stonewall Plant of Merck & Co. in Virginia, which is approximately the same size as the Cherokee Plant and operates similar chemical manufacturing processes.

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It was evident, after almost a year of operation, that it would be quite difficult, at the then current production levels, to maintain treatment efficiencies within the limits stipulated in the waste discharge permit utilizing only the existing treatment facilities.

Trickling Filter Studies

Experimental work has been successful at the Stonewall Plant using a trickling filter for the treatment of this type of waste (1). Accordingly, full-scale trickling filter facilities were being installed (2). Based on this information trickling filters were selected as the logical choice for an experimentation program at the Cherokee Plant.

After a few weeks of operation with the pilot plant, it became evident that trickling filters presented problems which were unanticipated and not easily solved. The major difficulty was an extremely prolific, gelatinous growth which would not slough, nor could it be induced to slough by the customary shock treatments. This growth was so profuse that ponding would occur in a matter of one to two weeks and the efficiency of the system would deteriorate from 70 per cent reduction to 20 per cent reduction, averaging about 40 per cent reduction.

The B.O.D. loading on the filters during these tests varied from 1 lb. per cubic yard of filter media to about 3.5 lb. per cubic yard. The hydraulic loading was varied from 15 m.g.d. to more than 25 m.g.d. in an effort to control the growth by hydraulic action.

Neither the B.O.D. loading nor the hydraulic loading had any great influence on the accumulation of growth. The only successful solution to the ponding problem was the complete replacement of the filter media, an obviously impractical answer. Due to the ponding condition, it was believed that trickling filters were not the answer to the treatment problem.

The decision was made to evaluate the activated sludge process as a method of treatment, and an experimentation program was initiated. It is the purpose of this paper to describe this work and the evaluation of the treatment as it was applied to this problem.

Laboratory Activated Sludge Studies

From the results obtained with the pilot-plant trickling filters, it was concluded that the waste material was not toxic to biological growth and that adequate nutrients were available. These are the primary requisites for any biological treatment system (3).

The factors affecting the operation of any activated sludge process are:

1. Concentration and chemical characteristics of the waste.
2. Concentration of active sludge in the aerator.
3. Contact time in the aerator.
4. Activity of sludge. (Measured by oxygen utilization rate.)

Of these several factors, only the sludge concentration and contact time are controllable in operation or design. The other factors are dependent on the nature of the waste. It is assumed that adequate oxygen is available and suitable pH conditions exist.

The laboratory program was established as a series of batch type tests in a 5-l. aerator. The aerator consisted of a cylindrical glass vessel, 6 in. in diameter and 16 in. tall.

Air dispersion was accomplished with two 3-in. diameter air turbines rotating at 600 r.p.m. These turbines were placed on a common shaft approximately 3 in. apart with the bottom impeller 1½ in. from the bottom of the vessel. Air was introduced under the bottom impeller.

A modification of the chemical oxygen demand procedure described by Porges *et al* (4) was used in these studies. Essentially this procedure consisted of heating the sample to 160° C. in 5 min. with sulfuric-phosphoric acid dichromate solution using silver sulfate as a catalyst. The sample was then cooled, diluted and titrated with standard ferrous ammoium sulfate using ferroin indicator. The use of silver sulfate as a catalyst was a modification proposed by the laboratories of the Atlantic Refining Co. The D.O. determinations were made polarographically, using a rotating platinum electrode in conjunction with a saturated calomel electrode (5).

Initial studies on the polarographic determination of dissolved oxygen were conducted with a dropping mercury electrode. Complete polarograms were determined for various process streams. These polarograms served as a basis for selection of the operating voltage, and they showed that it was not necessary to employ maxima suppressor. The dropping mercury electrode is quite suitable for determination of dissolved oxygen in samples of low suspended solids; however, it is not suited for samples such as aerator mixed liquors which may contain 1,000 to 5,000 p.p.m. suspended solids.

The most serious failing of the dropping mercury electrode is in the determination of the oxygen utilization rate of aerator mixed liquor. It was found that the sludge would settle rapidly when a sample of mixed liquor was placed in the cell of the dropping mercury electrode. As the sludge lost contact with the liquid, it could no longer obtain oxygen, and, therefore, the oxygen utilization rates thus determined were not linear with time.

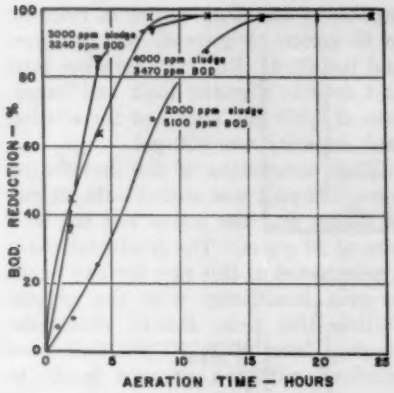

FIGURE 1.—The B.O.D. removed—aeration time relationship for several sludge and B.O.D. concentrations.

An attempt was made to overcome this difficulty by agitating the contents of the cell. This also failed because the mercury drops were disturbed when sufficient agitation was used to attain satisfactory suspension of the sludge, resulting in erratic results. Therefore, the rotating platinum electrode was used since this method would permit simultaneous sludge agitation and dissolved oxygen determinations.

Laboratory Studies

A series of batch type test runs was made with the 5-l. aerator using various sludge concentrations to determine the B.O.D. removed-aeration time relationship. The results of three typical tests are shown in Figure 1.

Since the data did not plot as expected, it was replotted as shown in Figure 2 using the loading factor versus the per cent reduction. For all practical purposes the three curves shown in Figure 2 are the same. Deviation is attributable to experimental error.

Several other test runs were made using various concentrations of B.O.D. and sludge. The sludge solids and B.O.D. concentrations in these runs varied from 1,050 p.p.m. to 5,000 p.p.m. for the sludge and from 730 p.p.m. to

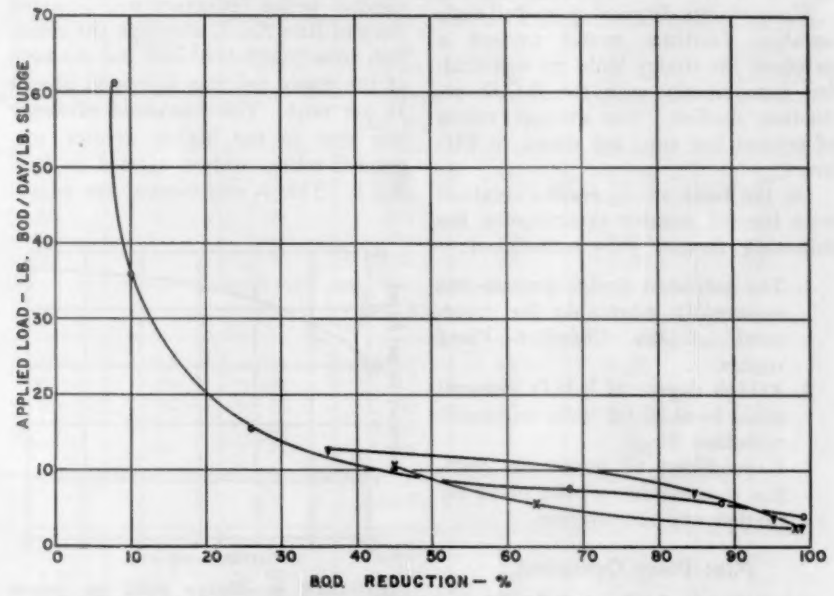

FIGURE 2.—Laboratory relationship between applied B.O.D. load and per cent B.O.D. reduction.

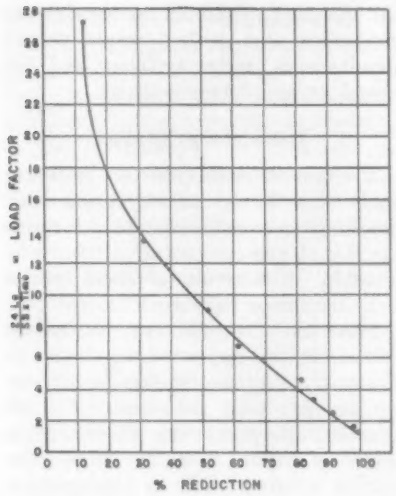

FIGURE 3.—Load factor versus average per cent B.O.D. reduction.

3,600 p.p.m. for the B.O.D. The seed sludge used was obtained from the Lancaster, Pa. sewage treatment plant. The results of these tests are averaged in Figure 3.

Since sludge disposal from full-scale aeration facilities would present a problem, the sludge build up was studied concurrently with the B.O.D. reduction studies. The average values of several test runs are shown in Figure 4.

On the basis of the results obtained with the 5-l. aerator experiments, the following factors were established:

1. The activated sludge process was apparently adaptable for treatment of the Cherokee Plant wastes.
2. A high degree of B.O.D. removal could be obtained with reasonable retention times.
3. Expenditure of funds was justified to construct a pilot plant for further experimentation.

Pilot Plant Operation

After the evaluation of the laboratory 5-l. aerator data, a pilot plant capable of handling wastes at rates up to 15 g.p.m. (Figure 5) was designed and installed. Both the aeration tank and aerobic digestor tank had capacities of 3,000 gal. each and the settling tank capacity was 600 gal.

With completion of the installation work, the unit was seeded with 20 gal. of sludge and the waste was fed at a rate of 10 g.p.m. The treatment plant was operated at this rate for two weeks to gain familiarity with the system. During this time, B.O.D. reductions ranging from 60 to 90 per cent were obtained with no attempt made to control the process.

An intensive 24-hr. study was started to obtain necessary data for full-scale plant design. This study was divided into three separate runs utilizing various feed rates and retention times. The various manufacturing process wastes were sampled at 2-hr. intervals during the study, but the results shown (Table I) are reported as daily averages.

It will be noted that in Run No. 2, slightly better reduction was obtained than in Run No. 1, although the retention time was halved and the strength of the waste fed was increased almost 18 per cent. This increased efficiency was due to the higher aerator suspended solids content carried in Run No. 2. This is evidence of the adapt-

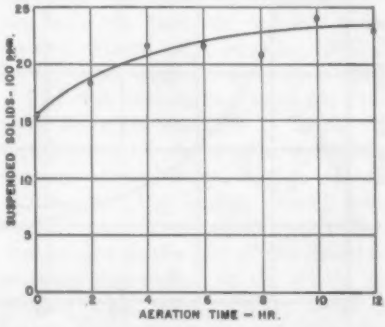

FIGURE 4.—Sludge build up versus aeration time expressed as p.p.m. suspended solids increase.

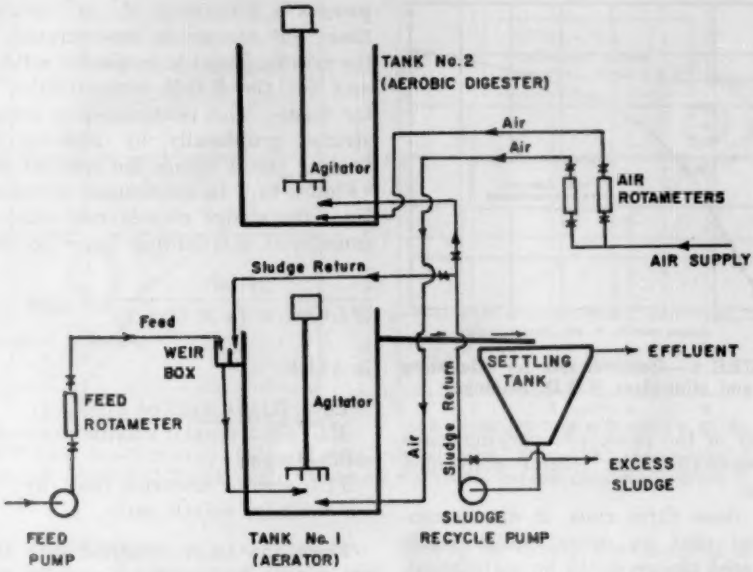

FIGURE 5.—Flow diagram of the activated sludge pilot plant.

TABLE I.—Activated Sludge Pilot-Plant Data

pH	Retention Time (hr.)	Air Flow (c.f.m.)	B.O.D. (p.p.m.)	B.O.D. Reduction (%)	Aerator Solids (p.p.m.)	Oxygen Utilization (p.p.m./hr.)
			Run No. 1			
8.9	5	43	2,065	89	3,276	44
7.9	5	43	1,785	88	2,750	35
6.1	5	43	931	83	2,093	31
6.4	5	43	936	90	2,604	37
7.3	5	43	1,429	88	2,680	37
			Run No. 2			
7.9	2.5	89	3,157	91	3,125	53
8.4	2.5	68	1,157	93	3,488	89
7.9	2.5	76	1,790	93	4,408	89
8.2	2.5	89	2,726	87	4,592	199
8.1	2.5	74	1,684	91	3,903	108
			Run No. 3			
6.5	2.5	89	3,157	53	2,056	135
6.2	2.5	96	2,510	50	1,780	169
6.8	2.5	96	1,929	63	1,827	125
6.5	2.5	94	2,532	57	1,888	143

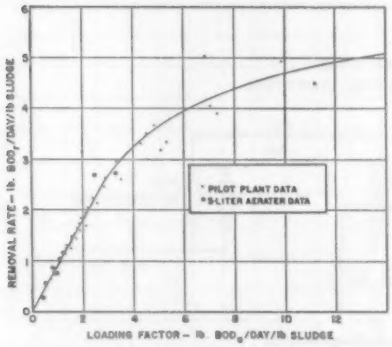

FIGURE 6.—Removal rate for laboratory and pilot-plant B.O.D. loadings.

ability of the process to varying loads by controlling the aerator suspended solids.

In these three runs, it was demonstrated that an operationally stable activated sludge could be maintained, using all the wastes from the manufacturing facilities. The only modification to the raw waste during this period of operation was to maintain a pH value between 6 and 8. Additional data taken at various times after this experimentation period have shown the same general results during an operational period of several months.

The effect of temperature on the system has not been fully evaluated, although it was demonstrated that a temperature of about 20° C. could be maintained in the exposed aeration tank during the winter.

The process was shown to be controllable and several degrees of treatment could be obtained. The low B.O.D. reduction shown in Run No. 3 was the result of a deliberate effort to obtain data in the lower efficiency ranges by reducing the aerator suspended solids and increasing the B.O.D. loading at low retention times, and does not represent failure of the system.

Evaluation of Data

The efficiency and degree of treatment of wastes by the activated sludge process is a function of: (a) aeration time; (b) the solids concentration in the aerator (volatile suspended solids); and (c) the B.O.D. concentration of the waste. This relationship is demonstrated graphically by plotting the loading factor versus the removal rate (Figure 6). In continuous operation, since the sludge recycle rate must be considered, this loading factor becomes

$$\frac{24\ La}{MLVSS \times T_h \times (1 + r)}$$ versus per cent removal

in which:

La = B.O.D. applied (p.p.m.);
$MLVSS$ = aerator volatile suspended solids (p.p.m.);
T_h = aerator retention time (hr.);
r = sludge recycle ratio.

From the curve obtained with this graphical relationship (Figure 7), it is possible to predict the performance of the activated sludge process for any given set of conditions of suspended solids, retention time, or B.O.D. loading. Furthermore, it is possible, within limits, to establish these conditions to produce any desired degree of treatment.

Two other important relationships in the evaluation of the activated sludge process are shown in Figures 8A and

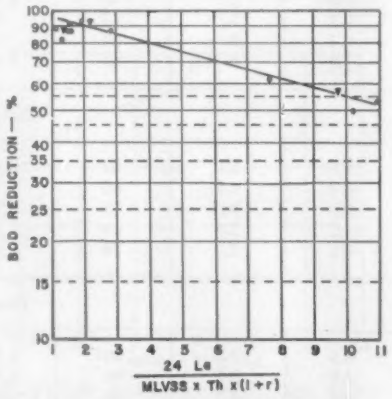

FIGURE 7.—B.O.D. reduction and loading relationship.

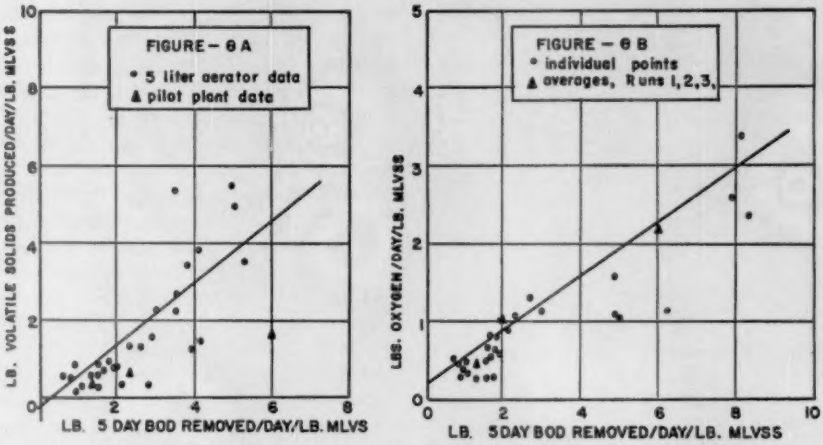
FIGURE – 6 A
• 5 liter aerator data
▲ pilot plant data
LB. VOLATILE SOLIDS PRODUCED/DAY/LB. MLVSS
LB. 5 DAY BOD REMOVED/DAY/LB. MLVS
FIGURE – 6 B
• individual points
▲ averages, Runs 1, 2, 3,
LBS. OXYGEN/DAY/LB. MLVSS
LB. 5 DAY BOD REMOVED/DAY/LB. MLVSS

FIGURE 8.—Activated sludge process relationships. Figure 8A represents sludge produced versus B.O.D. removed. Figure 8B represents oxygen consumed versus B.O.D. removed.

8B. These graphs show the relationship between the pounds of B.O.D. removed and the oxygen used or the sludge produced. When the pilot-plant data were analyzed for the preparation of Figure 8A, they failed to agree with known plant operational data and did not balance properly on calculation of the theoretical material balance.

It was believed that the sludge bulking, the inadequate measurement of sludge withdrawal and the inherent difficulty of taking sludge inventory in the system caused an error of such magnitude that the sludge production quantities derived were useless for computation. Hence, the production values obtained from the laboratory aerator data were used for the derivation of Figure 8A. The basis for this data was shown previously in Figure 4.

The principal basis for the design and the operating characteristics of an activated sludge treatment plant are revealed by the data presented in Figures 7 and 8.

Aerobic Digestion

Sludge, produced as a result of B.O.D. removal by biological methods, creates a major disposal problem. Among the standard methods of sludge disposal are: (a) filtration and incineration, and (b) anaerobic digestion.

The filtration and incineration of raw sludge was investigated, and rejected because the sludge was difficult to filter and incineration was an expensive method of solids destruction. Anaerobic digestion was attempted but proved unsatisfactory when the rate of decomposition of solids was determined to be too slow for practical application. In addition, the foul, putrefactive odors from this method might create an atmospheric pollution problem.

Because of endogenous respiration, aerobic digestion was considered as a partial answer to the solids disposal problem. Experimentation performed on aerobic digestion of settled sludge demonstrated that in six days of digestion a substantial destruction of solids and B.O.D. was obtained. The remaining solids were stable and produced no odor after standing for several weeks.

When the per cent solids reduction versus time in days is plotted (Figure 9), it may be predicted that with the aerobic digestion of the sludge a 55 to 60 per cent reduction of solids may be

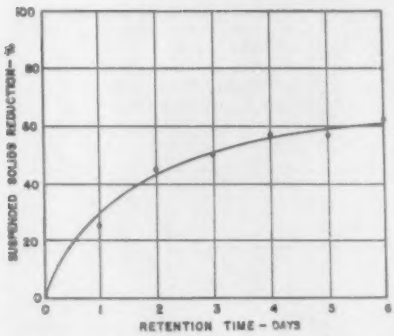

FIGURE 9.—Aerobic digestion of sludge.

obtained in 4 to 6 days. On the basis of these experiments, the volatile suspended solids were reduced 25 per cent per day and the oxygen utilization rate was 14 p.p.m. per hour per 1,000 p.p.m. of volatile solids.

Design Basis

Before any design work could be started on the waste treatment facilities, it was necessary to determine the anticipated flow and B.O.D. loading.

If the problem under investigation is a new waste treatment plant with no background of operating experience, the flow and B.O.D. loading can be determined only by careful estimate. However, if a fairly long, stable operating record is available, the determination of these factors is fairly simple. Let it be assumed that the latter case applies.

The first step is to prepare probability charts for the waste volume, B.O.D. load in pounds, the B.O.D. concentration, and any other known factors affecting the treatment plant design. These charts (Figures 10, 11 and 12) would be based on the analysis of past manufacturing plant performance at various production levels. In selecting flow and B.O.D. load values for the design of the waste treatment plant, the 90 per cent probability values were used rather than average values, since averages do not indicate a true picture of the frequency and magnitude of the variations.

The 90 per cent probability values of load and flow, give the following basic treatment plant design criteria:

Volume of raw wastes (Q) = 0.840 m.g.d.

B.O.D. discharge to receiving stream = 4,000 lb./day (assumed).

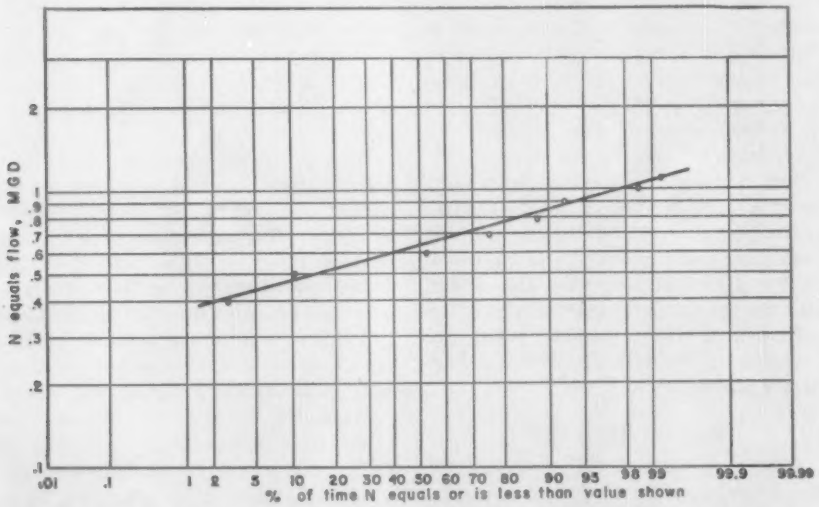

FIGURE 10.—Probability chart of waste flow based on six months of operation.

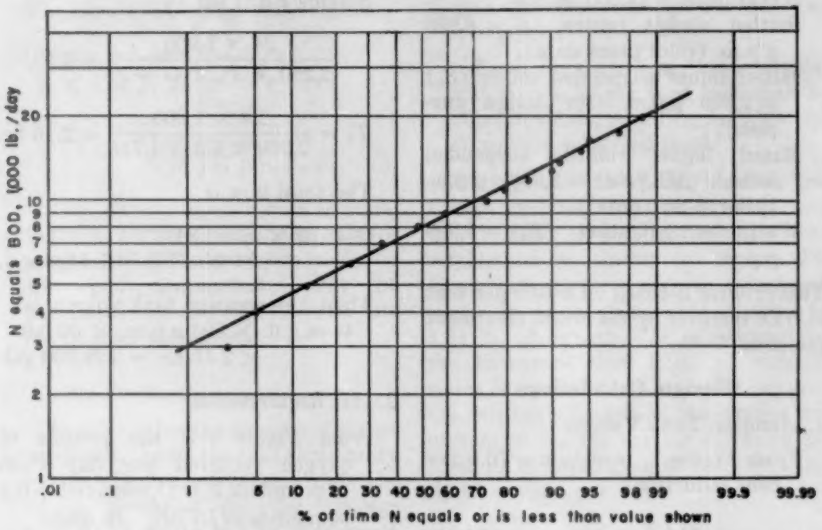

FIGURE 11.—Probability chart of waste load (B.O.D.) based on six months of operation.

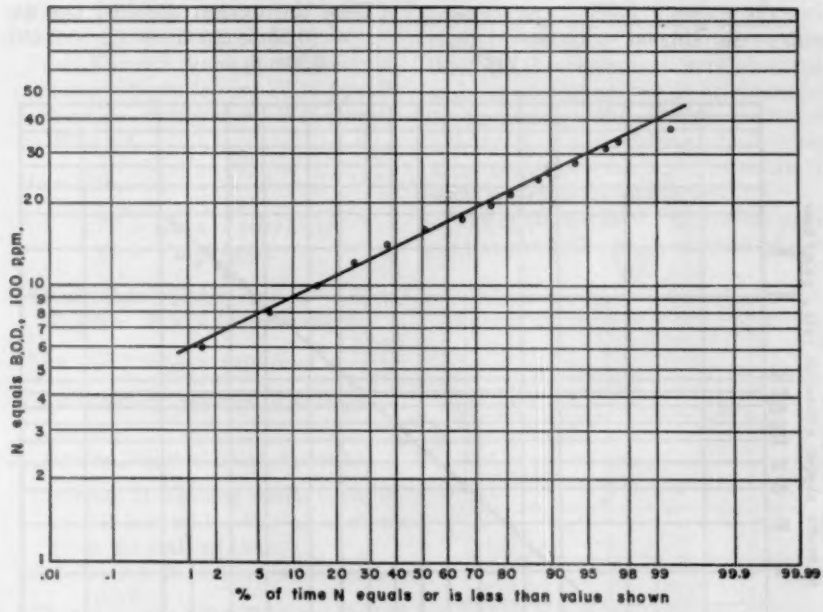

FIGURE 12.—Probability chart of waste concentration (B.O.D.) based on six months of operation.

B.O.D. load = 13,500 lb./day.
Settled sludge return (S_r = 6,000 p.p.m. (pilot plant data).
Mixed liquor suspended solids (S_a) = 2,500 p.p.m. (for design purpose).
Mixed liquor volatile suspended solids ($MLVSS$) = 2,250 p.p.m. (pilot plant data).
Waste concentration (L_a) = 1,900 p.p.m.

This criteria is based on a 70.5 per cent B.O.D. removal by the waste treatment facilities.

Design Calculations

1. *Aeration Tank Volume*

From Figure 7, to obtain a 70.5 per cent reduction,

$$\frac{24\,La}{MLVSS \times T_h \times (1 + r)} = 5.5 \quad (1)$$

The recycle ratio (r) equals

$$\frac{Sa}{Sr - Sa} = \frac{2,500}{6,000 - 2,500} = 0.715 \quad (2)$$

Solving Eq. 1 for T_h

$$\frac{24 \times 1,900}{2,250 \times T_h \times (1 + r)} = 5.5$$

$$T_h = \frac{24 \times 1,900}{2,250 \times 5.5 \times 1.715} = 2.15 \text{ hr.}$$

The total flow is

$$Q + (Q \times r) = .84 + (.84 \times .715) = 1.44 \text{ m.g.d.}$$

Then, the aeration tank volume is
1.44 m.g.d. × 694 g.p.m. × 60 min. × 2.15 hr. = 128,900 gal.

2. *Air Requirements*

From Figure 8 B the pounds of oxygen required per day = .35 × pounds of B.O.D. removed + 0.2 × pounds $MLVSS$. In which

.35 = slope of the curve and
.2 = Y intercept.

Then the oxygen required per day = (0.35 × 9,500) + (.2 × 2,450) = 3,810 lb./day.

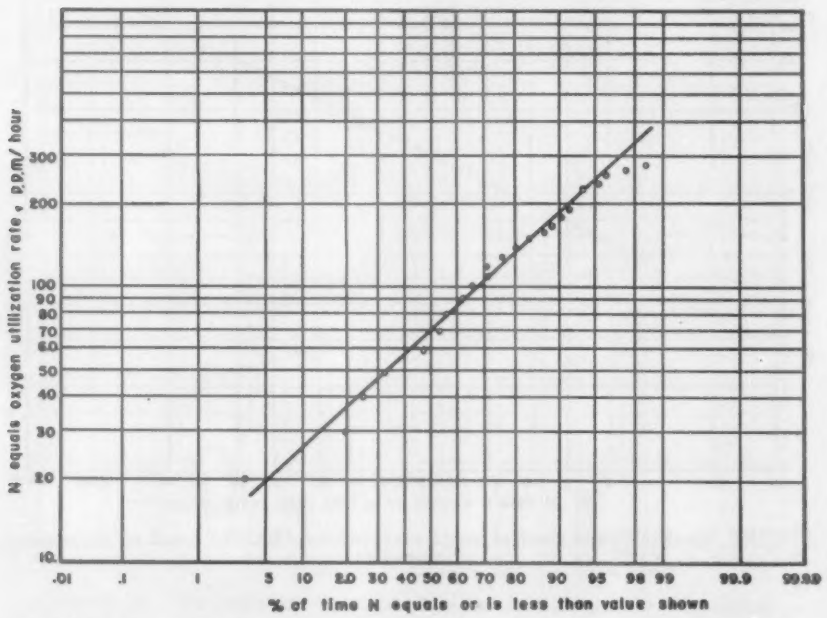

FIGURE 13.—Probability chart of oxygen utilization rate based on pilot-plant data.

The oxygen utilization rate equals

$$\frac{\text{Oxygen per day (lb.)}}{V \times 8.34 \times 24 \text{ hr.}} = \frac{3{,}810}{.129 \times 8.34 \times 24} = 147 \text{ p.p.m./hr.} \quad (3)$$

This oxygen uptake rate is equal to the 83 per cent probability rate for oxygen utilization as shown in Figure 13, which demonstrates that the calculated rate and experimental rate are in good agreement.

3. *Sludge Produced*

In calculating the amount of sludge produced, the 50 per cent probability values will be used because only the sludge produced by the average amount of waste will be withdrawn each day. A 90 per cent B.O.D. reduction will be obtained with an 8,000 lb. per day loading.

Pounds B.O.D. removed = 8,000 × .90 = 7,200 lb per day.

From Figure 8 A the pounds of volatile solids per day = .77 × pounds B.O.D. removed − .2 × pounds *MLVSS*.

In which,

.77 = slope of curve and
.2 = *Y* intercept.

Then the volatile solids = .77 × 7,200 − .2 × 2,450 = 5,060 lb.

The total sludge produced = $\frac{5{,}060}{.9}$ = 5,630 lb. per day (dry basis).

4. *Aerobic Digestion Tank Volume*

Material to digestor totals 5,630 lb. of 0.6 per cent solids as a slurry from the settling tank.

A 5-day retention time is assumed.

Then, $\frac{5{,}630}{.006}$ = 938,000 lb. per day of slurry = 112,500 gal. per day.

The aerobic digestion tank volume is 112,000 gal. per day × 5 days = 563,000 gal.

5. *Air Requirements for Aerobic Digestion*

From Figure 9, five days retention will give a 59 per cent reduction in volatile solids, resulting in 2,980 lb. of volatile solids oxidized.

During the pilot-plant studies, the sludge chemical oxygen demand and the volatile solids relationship was established as shown in Figure 14. From this curve it may be calculated that 1 lb. of volatile solids is equal to 1.34 lb. of oxygen. It is reported in the literature that 1 lb. of volatile solids is equivalent to 1.4 lb. of oxygen. On this basis therefore, the oxygen requirement for the aerobic digestion is: 2,980 lb. of volatile solids × 1.4 = 4,180 lb. of oxygen per day.

Conclusions

It is the opinion of the authors that the activated sludge process is quite superior to the other methods of treatment investigated for this type waste. Good B.O.D. reductions were obtained, and the process proved to be consistent in operation. The sludge recovered quickly to a high degree of efficiency after attempts were made to poison it with known toxic substances.

The operation of the treatment process was not difficult. Nontechnical per-

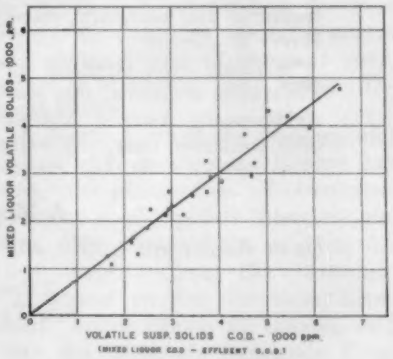

FIGURE 14.—Relationship between the C.O.D. of the volatile suspended solids and the mixed liquor suspended solids based on pilot-plant data.

sonnel learned the basic principles of operation within a few weeks. Technical supervision was required, but the responsibility for 24-hr. operation could be delegated to the nontechnical foremen. Therefore, from the standpoint of ease and efficiency of operation, the activated sludge process was the best answer found for the fine chemical waste disposal problem at the Cherokee Plant.

Acknowledgments

The authors express their sincere appreciation for the assistance and encouragement furnished by: J. N. Millar, Cherokee Plant Manager; Dr. R. A. Hoffman, Cherokee Research Manager; J. Lago, Research Chemical Engineer; R. G. Zimmerman, Staff Engineer; J. H. Morris, Central Engineering Division; F. E. Reimers, Central Research Division and the several laboratory technicians who worked on the project.

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AWWA NEWS

The American Water Works Association, friendly allies of hydrowasters, recently announced officer nominations and honors for 1956.

Nominated for 1956 president was Paul Weir, General Manager, Atlanta Water Works, Atlanta, Ga. Mr. Weir is presently serving as Vice-President of AWWA. Professor Fred Merryfield, Sanitary Engineering Professor, Oregon State College, Corvallis, Ore., received the nomination for Vice-President. Continuing as treasurer and secretary, respectively, are William W. Brush and Harry E. Jordan.

William John Orchard, an active participant in AWWA and Federation activities, was elected to receive the Harry E. Jordan Achievement Award. Not an annual selection, this award has been conferred only once before, on Abel Wolman in 1952.

A Moving Date

On or about April 1, 1956, **AWWA** headquarters will be moved to:

2 Park Avenue
New York 16, N. Y.

Stream Pollution

THE SEWERAGE AUTHORITY APPROACH TO STREAM POLLUTION CONTROL *

By Robert S. Shaw

Chief Public Health Engineer, New Jersey State Dept. of Health, Trenton, N. J.

New Jersey municipalities and industries are spending a total of approximately $250 million in their current construction programs for sewage collection and treatment and industrial waste treatment. The word "current" is used in this instance to describe construction which has been completed within the last two years or which, in all probability, will be completed within the next three years. Sewerage authorities, relatively new instruments of government, are responsible for more than 50 per cent of the current construction program in New Jersey.

The current construction program has reached its high level despite the fact that already there are more than 400 treatment plants, including more than 100 industrial waste treatment plants, in the State. The impetus for the program is derived from 4 principal facets of the stream pollution control problem in New Jersey.

Reasons for Program

First, it is significant that more than 30 per cent of the sewage treatment plants in New Jersey were constructed prior to 1920. More than 50 per cent of the sewage treatment plants are more than 25 years old. Also, more than 50 per cent of the industrial waste treatment plants are more than 15 years old. These statistics show that in the period following World War II New Jersey found itself with a large number of overloaded and antiquated sewage and industrial waste treatment plants.

A second significant facet of the problem is population density of New Jersey, which is second only to that of Rhode Island. This condition has obliged the State Department of Health to require more intensified sewage treatment methods in locations where lesser degrees of treatment formerly were considered adequate.

A third significant facet, at least in terms of treatment plant numbers, has been the widespread adoption of small "package" treatment plants and miniature conventional plants to serve realty subdivisions, in and beyond the fringe areas. It has become a trend in the larger subdivisions in New Jersey to construct sanitary sewers and sewage treatment plants in preference to individual home sewage disposal facilities.

A fourth facet of the problem, and a phase of tremendous significance, has been the construction of treatment plants to discharge into interstate waters. This phase of the program derived impetus from the Interstate Commission on the Delaware River Basin, better known as Incodel, and from the Interstate Sanitation Commission operating in the New York City metropolitan area. Little prog-

* Presented at 28th Annual Meeting, Federation of Sewage and Industrial Wastes Assns.; Atlantic City, N. J.; Oct. 10–13, 1955.

ress was possible and little was accomplished in these interstate areas until Incodel and the Interstate Sanitation Commission were established.

Sewerage Authority Authorization

In order to provide a concept of how sewerage authorities have fitted into the stream pollution control problem in New Jersey, a brief review of highlights of the general sewerage authorities law of the State is necessary. It should be made clear that the term "sewerage authority" does not encompass such organizations as the Passaic Valley Sewerage Commissioners, or the many joint meetings which are operating in the sewerage field in New Jersey. The latter agencies were predecessors to sewerage authorities in this State. They were, within limits, endeavors of government to accomplish some of the purposes of sewerage authorities.

The general sewerage authorities law in New Jersey is known as Chapter 138 of the Laws of 1946. It was enacted to extend and implement the State policy established in 1899 toward relief of waters of New Jersey from pollution. The law seeks to accomplish its purposes by authorizing the creation of sewerage authorities empowered to provide sewerage services at the expense of users without increasing the burden of general taxation.

A sewerage authority is an agency of government representing one municipality or a part thereof, two or more contiguous muncipalities, a county or a part thereof. Sewerage authorities are created by local, county or municipal government. When more than one municipality is involved, an authority is created by the concurrent action of the interested municipalities, or by the action of county government. Once created, sewerage authorities are empowered to operate as independent agencies of government.

In the language of the law a sewerage authority is a "public body politic and corporate" constituting a political subdivision of the State, established as an instrumentality exercising public and essential governmental functions to provide for the public health and welfare. An authority may sue and be sued. An authority may acquire, hold and use, and dispose of service charges or other revenues. It may acquire by purchase, gift, condemnation or otherwise real property and easements necessary or useful for its purposes. It may provide for and secure the payment of any bonds and the rights of the holders thereof. It may enter into contracts for the purposes of the authority.

Financial Details

A sewerage authority is authorized to charge and collect rents, rates, fees or other charges usually referred to as service charges. The law establishes limitations within which an authority must work in establishing rate schedules. The law requires that service charges be as nearly uniform as practicable throughout the district served for the same type and amount of service. The law also establishes various means for setting up rate schedules.

The most significant provision of the law in relation to charges is the requirement that the revenues of a sewerage authority will at all times be adequate to pay all expenses of operation and maintenance of the sewage system including reserves, insurance, extensions and replacements; to pay punctually the principal and interest on any bonds; and to maintain such reserves or sinking funds therefor as may be required by the terms of any contract of the sewerage authority, or as may be deemed necessary or desirable by the sewerage authority. The law also makes provision for the revision of service charges from time to time as deemed necessary.

It is important to note that the bonds issued by a sewerage authority do not

constitute in any way a debt or liability of the State or any unit of local government. Sewerage authorities are not permitted to incur any indebtedness on behalf of the State or any county or municipality.

Powers of an Authority

After a sewerage authority has been created, no sewage disposal plant or other facilities for the collection, treatment or disposal of sewage arising within the district of the authority may be constructed without the approval of the authority. Sewerage authorities also are granted police powers over stream pollution within their district. They share these powers with the State Department of Health.

The foregoing brief summary of the provisions of the sewerage authority law omits many technical details required to establish a sewerage authority. Perhaps the most significant conclusions to be drawn from this summary are that sewerage authorities are agencies of government, and that they represent delegation of authority by local, county or municipal government in the sewerage field.

Political Considerations

The sewerage authorities law was borne of economic and political expediency. The local tax burden had become great, and legal debt limits had been approached or exceeded in many municipalities; thus it was deemed advisable by fiscal authorities that some means should be provided for establishing, operating and maintaining sewerage facilities on a fiscal basis independent of other local government services.

Obviously there also were political implications involved. During the late 1930's and early 1940's the State Department of Health in New Jersey was very active in exercising its police powers in stream pollution control. It was during the latter part of this period that activity was started in the interstate area. Actions brought by the State Department of Health and the interstate agencies obligated local government and industry, immediately after World War II, to proceed with the design and construction of adequate sewage and industrial waste treatment facilities.

In some areas of the State, joint action by local governments and industry was indicated. Such joint action involved from 2 to 50 or more municipalities in various situations. This obviously led to political problems and furthered consideration of legislation such as the Sewerage Authorities Law. It is generally recognized by those familiar with joint action by local governments, that difficulties are encountered in securing concurrent action for joint municipal projects. Obviously the problem becomes much greater as the number of municipalities involved increases. The Sewerage Authorities Law was designed to facilitate operation in such joint action. Once created a sewerage authority is empowered to proceed with the solution of a sewerage problem independent of the various local governments which may have been involved in its creation.

New Jersey Experience

What has been the New Jersey experience with sewerage authorities? Have they been a success? Are they a good idea? Will they endure? It is too early, undoubtedly, to provide conclusive answers to all these questions. Some indications, however, seem clear from brief experience.

There are approximately 30 sewerage authorities operating in New Jersey today. The majority of these authorities represent one municipality or a part thereof. Others represent several municipalities, or large segments of whole counties involving 50 or more municipalities. There has been but one instance of the abandonment of a sew-

erage authority, and this was in one of the smaller municipalities of the State.

As indicated earlier, sewerage authorities have been successful in instituting within only a few years sewerage construction in New Jersey costing more than $125 million. Much of this progress has been made possible because of the facility of a sewerage authority in accomplishing its ends with a minimum of political interference and a minimum of red tape. The Sewerage Authorities Law has made it possible for truly interested local citizens to accomplish sewerage projects in a relatively short time. It might also be said that the law has made it possible for elected local officials to relieve themselves of many of the burdens involved in sewerage projects, by delegating the work to another agency.

Nothing has been accomplished by sewerage authorities that could not have been accomplished by established local government. The question of importance is, would local government have done the job? The New Jersey experience indicates that local government might ultimately have done the same job, but undoubtedly many more years would have been required to accomplish it.

The case against sewerage authorities to date appears to be somewhat as follows:

1. Sewerage authorities represent an undue delegation of authority to a relatively independent agency, and to some degree represent a departure from strictly democratic government.

2. Sewerage authorities can be and in some instances undoubtedly have been expensive. The interest rate on sewerage authority bonds sometimes may be higher than would have prevailed in the instance of municipal financing, because of the fact that the credit of the municipality or other local government unit is not involved in authority bond issues.

3. Sewerage authorities may deny the extension of their services if they believe such extension to be economically unsound, even though it might be indicated as desirable for the general welfare.

Whether or not sewerage authorities endure probably depends primarily upon the qualifications of the membership of authorities, and the members' determination to manage sewerage facilities in a business-like manner. There are some who believe that it is only a matter of time before appointment to a sewerage authority will be dependent solely on one's political loyalty. If this situation should develop, a short life must be anticipated for the sewerage authority movement. Fortunately, there is to date much indication of active interest in sewerage authorities by citizens of the highest calibre. If this situation prevails there is no reason to doubt that the sewerage authority movement will continue to be a real success in New Jersey.

JANUARY 1955 JOURNALS NEEDED

The increased growth of the Federation is reflected in the short supply of Journal back numbers. Recently the supply of the January, 1955 issue of SEWAGE AND INDUSTRIAL WASTES became depleted.

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PULP MILL POLLUTION IN ALBERNI HARBOR, BRITISH COLUMBIA

By Michael Waldichuk

Fisheries Research Board of Canada, Pacific Biological Station, Nanaimo, British Columbia, Canada

The original study of oceanography in Alberni Inlet was conducted by Tully (1) to predict the effect of pulp mill effluent discharge on the biotic environment of the inlet. The results of this study prompted industry to reconsider the process to be used in their pulp production. In place of the sulfite mill originally planned, from which it was evident a high degree of pollution would be discharged during the summer months, the company installed a sulfate (Kraft) mill. Since 1948, this mill has been discharging wastes from a production of up to 230 tons per day into the head of Alberni Inlet at the estuary of the Somass River. Following the first months of operation, water samples taken at a few strategic points showed no deficiency of dissolved oxygen (2). No complete survey of the inlet waters was made at that time.

A new management announced its intentions to expand the mill to a capacity of 500 tons per day of unbleached Kraft pulp and 500 tons of newsprint (mainly groundwood). It was desirable, therefore, to observe the effects of the discharge from the existing mill on Alberni Inlet water in order to predict the effect of the effluent from the planned expansion. This paper presents some of the results and considerations of the survey conducted in September, 1954.

Situated on the west coast of Vancouver Island, Alberni Inlet is connected to the Pacific Ocean by Barkley Sound. The harbor is relatively shallow with a maximum depth of 19 fathoms (114 ft.) shoaling into extensive tidal flats at the north end. The Somass River, discharging into the northeast end of the harbor, has been responsible for the tidal flats and marshlands in that area through deposition of its silt load. The condition of the water in the harbor is of concern from a fisheries point of view, in that substantial runs of several species of Pacific salmon and sea-run trout support a small commercial fishery as well as a popular sport fishery. The spawning migrations of most of these anadromous fishes to the tributary rivers of the Somass River occur during the period of low runoff in late summer, when the effects of pollution are at a maximum.

Survey Technique

The survey was conducted during the week, September 7 to 11, 1954. Samples for salinity, dissolved oxygen, pH and lignin determinations were taken at the surface, 6, 15, 30, 45, 60, 90 ft., and near bottom (or at as many of these levels as the depth permitted). Sampling bottles of the Fjarlie design (3) were used for subsurface samples. Dissolved oxygen and pH determinations were carried out on board ship within 24 hr. after the samples were taken. The procedure described by Tully (1) for shipboard dissolved oxygen analysis was used. A Beckman pH meter was employed for the pH determinations.

The nitrosolignin test for sulfite waste liquor, described by Pearl and Benson (4), was employed to determine Kraft waste concentration as

lignin. A Klett-Summerson colorimeter with No. 54 filter and 45 mm. cell was used to measure the color intensity. Color of the water and depth of light penetration were examined with a standard, white 30-cm. diameter Secchi disc. The bottom materials in the harbor were checked with a sounding lead and tallow by the standard method (5).

The survey period normally is a time of small runoff, but 1954 was an anomalous year with precipitation all during the summer. Consequently, the Somass River discharge computed from the sum of Stamp and Sproat Rivers did not reach its usual low point of less than 500 c.f.s.

Observed Characteristics

Distribution of properties at the surface in Alberni Harbor is illustrated in Figures 1 and 2. Salinities throughout the harbor, Figure 1, ranged from 5 to 10 parts per thousand (‰).

A very crudely determined distribution of effluent in the surface water is shown in Figure 1. The distribution of effluent was characterized by patchiness. High concentrations were found in thin patches along the waterfront. Generally the high concentration was separated from the well diluted mixture by a foam line. The concentration of effluent fell off to 2 per cent at the surface in less than one-half mile from the outfall. Smaller concentrations could not be measured satisfactorily by the technique used.

A surface oxygen distribution illustrated in Figure 2 combines data obtained from regular Stations L20, 1, 2, 3, 4, and 5 occupied on September 9, 1954, with surface samples collected

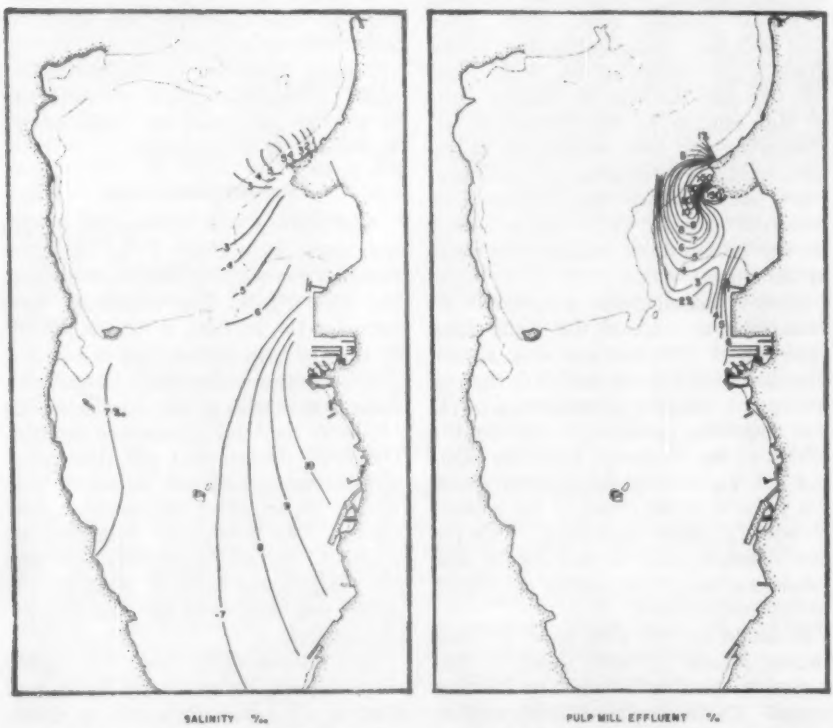

FIGURE 1.—Salinity and pulp mill effluent in the surface water of Alberni Harbor.

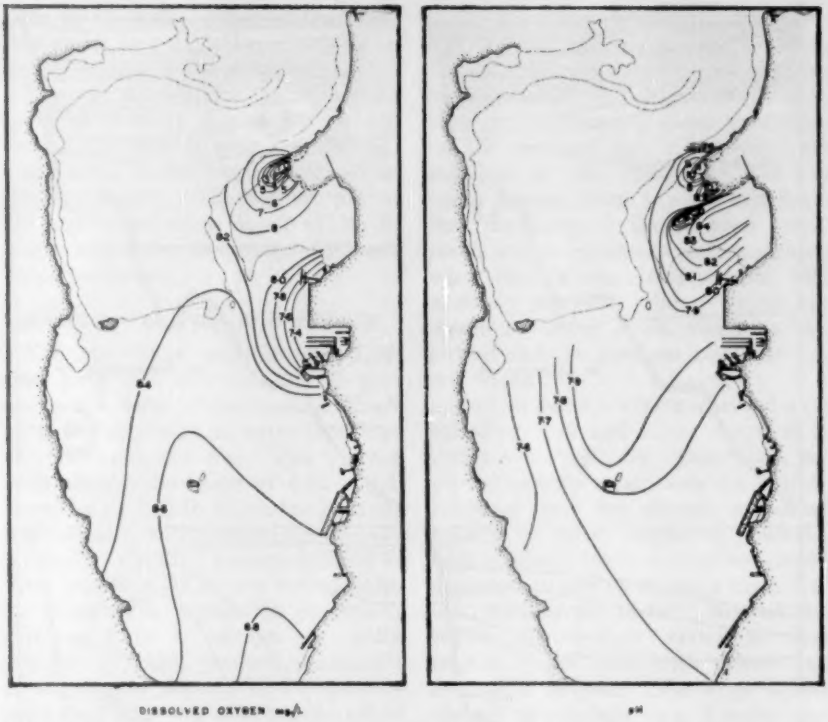

FIGURE 2.—Dissolved oxygen and pH values of the surface water of Alberni Harbor.

from the area surrounding the pulp mill outfall on September 10. There was a general increase of dissolved oxygen from the outfall area seaward. Except in the immediate vicinity of the outfall, dissolved oxygen concentration in excess of 5.0 mg. per liter occurred throughout the harbor.

The pH value distribution shown in Figure 2 generally corresponded to the dissolved oxygen trend in the main part of the harbor. In the outfall area, however, the pH value increased as the concentration of effluent increased. The effluent reaching the sea had a pH of 11.3, but this rapidly decreased as dilution occurred. Sea water is a solution of numerous buffering salts so that any acidic or alkaline constituents added are rapidly restored to near neutrality. Somass River water had a pH of about 7.5 which was generally lower than that of sea water. Consequently, in the area outside the outfall there were at least four factors which influenced the pH value: (a) the effluent with a high pH; (b) river water with a relatively low pH; (c) oxidation of the effluent reducing the pH; and (d) the buffering action of sea water with a pH value of about 8.0.

The vertical distribution of properties in the harbor is shown in Figure 3. The salinity chart, Figure 3a, shows the very pronounced gradient in salinity for the upper 15 ft., with a salinity at 20 ft. comparable to that of the deep water in Barkley Sound. The temperature chart, Figure 3b, followed a similar pattern, with a 12° F. gradient in the upper 30 ft. Water below 30 ft. depth, actually was cooler than the water at the same level in Barkley Sound. This suggests that the deep

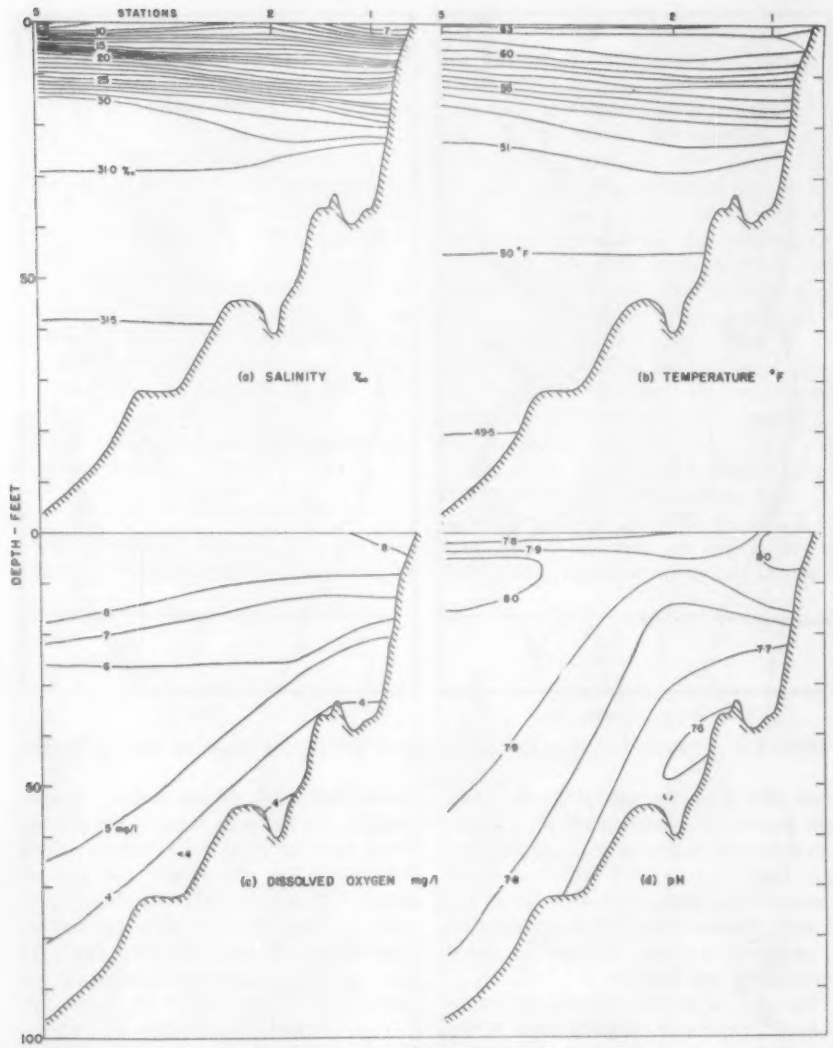

FIGURE 3.—Vertical distribution of characteristics in Alberni Harbor: (a) salinity, (b) temperature, (c) dissolved oxygen and (d) pH value.

water in the inlet had not been subjected to an increase in temperature by thermal conduction, currents or mixing from the time it first entered the deep zone in Alberni Inlet. In the meantime, the outside water had been receiving heat to considerable depth by wind and tidal mixing.

The oxygen distribution illustrated in Figure 3c shows the low oxygen concentration in the deep water (less than 4 mg. per liter). Slope of the iso-oxys upward to the northern end of the harbor indicates either movement of deep water toward the surface, to replace sea water being swept

away by the seaward flow of fresh surface water, or a depletion of oxygen in a constant depth above the bottom.

The pH distribution in Figure 3d shows a distinct conformity to the oxygen distribution, with a similar upward trend in the isopleths of pH in the deep water. The highest pH of 8.0 was found at a depth of 10 ft. where relatively uncontaminated sea water occurred.

Character of Pulp Mill Effluent

The MacMillan and Bloedel, Limited, pulp mill in Port Alberni produces 230 tons of unbleached Kraft pulp per day with a waste discharge of 28,000 gal. per ton. This gives a total effluent discharge of 6.44 m.g.d. Based on its B.O.D. which has been experimentally determined to be 150 p.p.m. (6) the total oxygen demand of these wastes is 35 lb. per ton of pulp. In terms of a population equivalent given as 0.168 lb. oxygen per capita per day (7), this production amounts to the oxygen demand of 208 persons per ton of pulp or a total population of 47,840 for the daily production.

The effluent from the pulp mill flows in an open channel through a fairly extensive meadow and marshland in its course from the mill site to the sea, receiving considerable aeration during its turbulent flow. On reaching the sea the effluent is at a temperature of about 80° to 90° F., and is completely devoid of dissolved oxygen.

Behavior of Effluent in Harbor

The effluent flows over the sea water in a thin (2 to 6 in.) layer immediately beyond the outfall. This was apparent in the large temperature gradient from the warm surface layer (86° F.) to the deeper water (68° F.) which could be measured in the length of an ordinary 10-in. thermometer. Mixing of this surface layer into the sea water occurs beyond a foam line region surrounding the outfall in a radius of about 100 yd.

The foam line marks a sharp gradient between the warm, dark brown, low-salinity and low-oxygen mixture on the outfall side and the dilute mixture approaching sea water conditions on the seaward side. Evidently the stability of the effluent on the sea water breaks down beyond this foam line boundary. Observations were made on an extremely calm morning when the sea was glassy smooth. On a windy day the mixing would undoubtedly occur much closer to the outfall and the gradient would be less noticeable.

Bottom materials were observed with sounding lead and tallow for lack of better sampling gear. Conditions improved rapidly away from the outfall. Residues from the effluent, consisting mainly of white chalk-like materials with strata of black organic substance, appeared to settle out into a layer 2 to 5 ft. thick at the outfall. Beyond this region the bottom materials were largely coarse sand with gravel and fragments of bark. The river mouth showed no evidence of mill waste particulate matter having accumulated on the bottom.

The color of the water changed from dark brown immediately outside the outfall, where the Secchi disc could be seen only to a depth of 1 ft., to a brownish-green in the middle of the harbor (with a Secchi disc reading of 19 ft.). In the saw mill and log boom area the water had a reddish-brown tinge. The improvement of water color and light penetration was rapid outside the foam line.

Discussion and Summary

There is no oxygen shortage in the surface water of Alberni Harbor except in the immediate vicinity of the pulp mill sewer outfall. The recommendation of Tully (1) to maintain the sewer discharge at the surface seems to have been a good one, inasmuch as advantage is taken of the

seaward flow, high dissolved oxygen in the surface water, as well as wind aeration. Deep water is already depleted of oxygen and would not support much oxidation of foreign material.

On the basis of work by Tully (1) the daily oxygen provided by the fresh water and sea water mixed in the upper zone at the discharge of the Somass River during the survey (1,245 c.f.s.) is 100,000 lb. The oxygen required for effluent oxidation is 8,000 lb., leaving 92,000 lb. in the water. This amounts to an average of 6.8 mg. per liter when spread over the entire harbor, a good check with the observed results and well on the safe side for fish requirements. Although the survey was conducted during the period of normally low Somass River discharge, the 1954 season cannot be considered as representative. Ordinarily much lower discharges are met during this period and far less oxygen is available.

Variations in pH are too small to be of any consequence to the fish. The buffering capacity of sea water maintains the pH close to 8.0 at all times. Only in the region of high effluent concentration and low dissolved oxygen were pH values much in excess of 8. It has been established that a tolerable pH range for most fish is 5.0 to 9.0 (8).

Observations of mill waste concentration have borne out results reported on model experiments using dye as a tracer (1)(9). The midharbor area in the path of the jet stream from the Somass River appears to be well flushed and relatively free of pollutant. Northwest and northeast corners of the harbor, however, are occupied by back eddies which tend to accumulate and stagnate the effluent.

It can be generally concluded that there is no serious condition of pollution in Alberni Harbor at the present time from a fisheries point of view. The possibility does remain, however, that it is the toxicity of the effluent in high concentration and not just its oxygen demand which may be of primary concern after expansion of the pulp mill.

Dissolved oxygen concentration and pH are normal everywhere except in the immediate vicinity of the mill sewer outfall. Discharge of mill wastes at the surface of the inlet instead of at depth prevents further depletion of the oxygen in the deep water already low in oxygen content as a result of natural causes. Concentration of the pulp mill effluent was quite high in the surface water at the mouth of the Somass River. This situation is probably too short-lived, occurring only on the flood tide and during calm weather, to be of any consequence to the fishery.

Acknowledgments

The data were collected from C.N.A.V. *Ehkoli* through the courtesy of the Pacific Naval Laboratory and the Pacific Oceanographic Group. MacMillan and Bloedel, Limited, have kindly permitted the use of their production data to characterize the mill effluent.

The author wishes to acknowledge the helpful suggestions provided by Dr. J. P. Tully throughout this work.

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MEETINGS OF INTEREST

1956

Apr. 13–16 **United States–Mexico Border Public Health Assn.,** Calexico, Calif.–Mexicali, Baja, Calif. Fourteenth Annual Meeting. Sidney B. Clark, M.D., Secretary, 243 U. S. Court House, El Paso, Texas.

Apr. 15–17 **Western Public Works Conference,** Oakland, Calif. General Chairman, John A. Morin, City Engineer, City Hall, Oakland, Calif.

Apr. 17–18 **Ninth Municipal and Sanitary Engineering Conference,** University of Florida, Gainesville, Fla. Dr. E. R. Hendrickson, College of Engineering, University of Florida, Gainesville, Fla. Conference theme—Atmospheric Pollution.

May 6–11 **American Water Works Association Diamond Jubilee Conference,** St. Louis, Mo. Harry E. Jordan, Secretary, AWWA, 2 Park Ave., New York 16, N. Y.

May 15–17 **11th Purdue Industrial Waste Conference,** Purdue University, Lafayette, Ind. Professor Don E. Bloodgood, School of Civil Engineering, Lafayette, Ind.

June 4–8 **American Society of Civil Engineers,** Knoxville, Tenn.

June 18–22 **Gordon Research Conference on Stream Sanitation,** Kimball Union Academy, Meriden, N. H. Dr. W. George Parks, Director, Gordon Research Conferences, University of Rhode Island, Kingston, R. I. Applications necessary by April 1; conference limited to 100 persons.

June 18–23 and 25–30 **Bioxidation of Organic Wastes,** Manhattan College, New York, N. Y. Assistant Professor W. Wesley Eckenfelder, Civil Engineering Dept., Manhattan College, New York 71, N. Y. Special course limited to 18 persons at each of two sessions. Registration fee $150.

SLIME FORMATION IN SEWAGE*

III. Nature and Composition of Slimes

By H. Heukelekian and E. S. Crosby

Rutgers University, New Brunswick, N. J.

Editor's Note: The third and final section of this paper comprises a report on the physical, chemical and biological composition of slime growths developing on surfaces submerged in sewage and in polluted waters. Parts I and II appeared in the December, 1955 and January, 1956 issues of This Journal.

Physical and Chemical Characteristics

The gross appearance of slime growths that developed on surfaces submerged in sewage generally was very similar. During primary film formation (the first 3 to 5 days), slimes usually were brown or greyish-brown in color. In aerobic environments the slimes remained brown or became green, due to algae growth. Few algae developed in slimes submerged in sewage, even in the open channel, because insufficient light penetrated through the sewage.

When the slime growths became thick, their color was greyish-black or black, both in aerobic and anaerobic environments; but in aerobic situations their surface appeared greyish to white, due to large numbers of stalked colonial protozoa. The color of the slimes appeared to vary with the dissolved oxygen content of the environment in which they formed. A high dissolved oxygen content appeared to be associated with brown slimes, and a low dissolved oxygen content or oxygen deficiency with growths grey to black in color.

Slimes submerged in sewage always were watery in consistency containing little of the filamentous material or matted growths usually found in trickling filters. The types and consistency of slime growths appeared to be related to the concentration of the medium in which the growths developed. Abundant and watery slimes, which readily sloughed off, were characteristic of all growth in liquids high in B.O.D. value; while dense, readily adhering slime growths were more characteristic of growth in liquids low in B.O.D. value (as in the receiving stream).

The odor of slimes was generally similar to that of the medium in which they formed. Slime films tended to become increasingly septic or foul in odor as they became older and thicker.

The moisture and ash content of slime samples was determined. Field samples of slime growths were used, since laboratory growth rates were too slow to develop sufficient quantities of slime for analysis. The moisture and solids values of slimes from four types of test plates, immersed for periods up to 56 days in the primary clarifier effluent channel, are summarized in Table I. These results show that the moisture content was similar for 20 samples of slimes from concrete, glass, glazed tile and transite surfaces. These values ranged from 91.3 to 97.0 per cent moisture and 19.4 to 32.9 per cent ash in the dry solids, the average values being 94.7 per cent and 24.1 per cent, respectively.

A comparison, Table II, of the moisture and ash contents of slime films grown under aerobic and anaerobic

* Paper of the Journal Series, New Jersey Agricultural Experiment Station.

TABLE I.—Moisture and Solids Content of Slimes Growth Under Aerobic Conditions

Days Immersed	Test Plate Material							
	Concrete		Glass		Tile		Transite	
	Moisture (%)	Ash[1] (%)	Moisture (%)	Ash[1] (%)	Moisture (%)	Ash[1] (%)	Moisture (%)	Ash[1] (%)
6	97.0	21.0	95.3	20.6	96.5	26.4	94.7	22.8
13	95.3	20.6	95.5	19.6	95.4	19.4	95.1	21.0
20	94.3	26.0	93.5	21.6	93.9	32.7	91.3	31.0
27	95.0	24.4	94.0	25.8	94.3	21.2	94.5	21.6
35	94.0	29.8	95.3	21.0	95.3	23.2	94.8	26.0
42	—	32.9	—	24.0	—	25.4	—	25.4
48	—	23.2	—	21.0	—	23.4	—	22.4
56	—	25.4	—	24.0	—	22.2	—	26.2
Av.	95.1	25.4	94.7	22.2	95.1	24.2	94.1	24.5

[1] Ash, per cent of dry solids.

conditions on concrete surfaces shows similar results, except that ash content was somewhat lower in the anaerobic environment than in the aerobic environment. Considering the variations found in a large number of analyses of slimes grown on test plates, no significance can be attached to the differences between aerobic and anaerobic environments.

The results of a typical analysis of a slime film formed on a surface submerged for a period of 20 weeks in settled sewage, are given in Table III. These data show that the moisture and solids contents were similar to those of recently formed slimes. The volatile solids content was slightly less, possibly due to accumulation of silt- and ash-forming materials. The total organic nitrogen and ammonium nitrogen present in the slime was lower than would be expected if the slime was primarily of biological origin; this finding indicates that, in addition to biological growths, there was deposition or adsorption of materials from the sewage. The grease content of this slime was similar to that of domestic sewage sludge. In general, slimes were similar in appearance and characteristics to primary sludge.

TABLE II.—Moisture and Solids Content of Slimes Growth Under Aerobic and Anaerobic Conditions[1]

Determination (%)	Aerobic[2] Range	Average	Anaerobic[3] Range	Average
Moisture	95.26–95.35	95.31	94.42–95.52	94.47
Total solids	4.65– 4.74	4.69	5.48– 5.58	5.53
Volatile solids	76.4 –76.9	76.6	79.8 –80.6	80.0
Fixed solids	23.1 –23.6	23.4	19.4 –20.2	20.0

[1] Immersion period, 57 days.
[2] Station 6, 6 replicate samples.
[3] Station 4, 4 replicate samples.

Biological Characteristics

Field Conditions

The slime growths developed under field conditions were examined micro-

TABLE III.—Analysis of Sewage Slime

Determinations[1]	Results
pH	6.8
Density at 20° C.	1.0122
Moisture, per cent of wet weight	95.4
Total solids, per cent of wet weight	4.6
Volatile solids, per cent of total solids	70.7
Ash, per cent of total solids	29.3
Total nitrogen, per cent of total solids	1.0
Ammonia nitrogen, per cent of total solids	0.5
Grease, p.p.m.	15.7

[1] Based on duplicate analyses of a single slime sample, collected from a Transite test surface immersed 20 weeks.

scopically to compare the microflora and microfauna present in anaerobic and aerobic locations, as well as those at low and high velocity stations. The object of these examinations was to determine the gross differences and relative abundance of the various types of organisms rather than to compile a detailed enumeration. Complete microscopical results are not given because of space limitations, but the following generalizations can be made.

The biological composition of sewage slimes developed on slides submerged in anaerobic and aerobic locations, namely in the primary effluent channel and the final effluent channel of a sewage treatment plant was similar during the first 1 to 3 days of growth. The few protozoa present during this period were mainly free-swimming flagellates and ciliates.

At the time of maximum slime accumulation (10 to 14 days), the predominant microfauna in the films from anaerobic locations were free-swimming flagellates. Free-swimming ciliates and amoeba were less frequent. A more diversified fauna was obtained in the slimes from an aerobic environment, the predominant organisms during this period being Vorticella, Carchesium, free-swimming ciliates and flagellates, small nematodes, and aquatic oligochaetes in descending order of frequency.

The bacterial flora in the slimes from the aerobic and anaerobic locations consisted of short and long rods and large coccoid cells. After the initial film was formed, filamentous sulfur bacteria (Beggiatoa and Thiothrix) were present in the films from anaerobic as well as aerobic locations. As the film developed, pseudopodial or finger-like growths of zoogleal bacteria, consisting of gram positive cocci, became abundant. Microscopically, the microflora and microfauna present in slime films grown in low and high velocity anaerobic environments were similar. Figures 1, 2, and 3 are photomicrographs of slimes (original magnification 970 ×, reduced about one-half).

Laboratory Conditions

Results of the microscopical examination of slimes formed in various laboratory methods of submergence also are summarized.

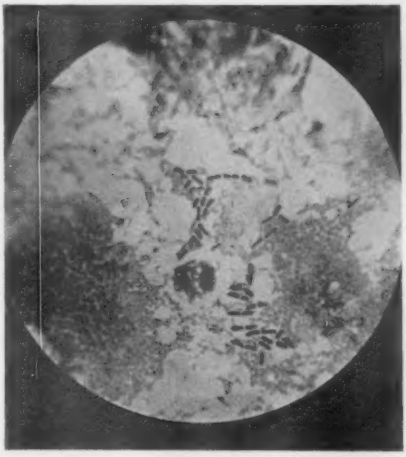

FIGURE 1.—Photomicrograph of aerobic slime film after two days of growth in final clarifier effluent (gram stain).

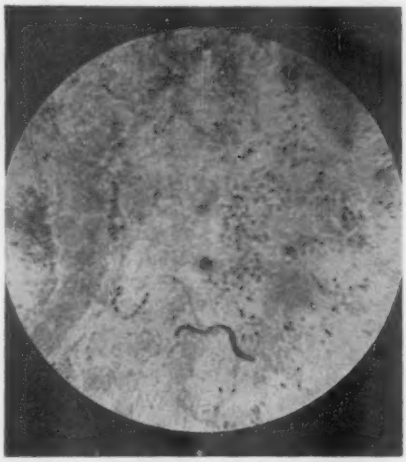

FIGURE 2.—Photomicrograph of anaerobic slime film after 8 days of growth in primary clarifier effluent (gram stain).

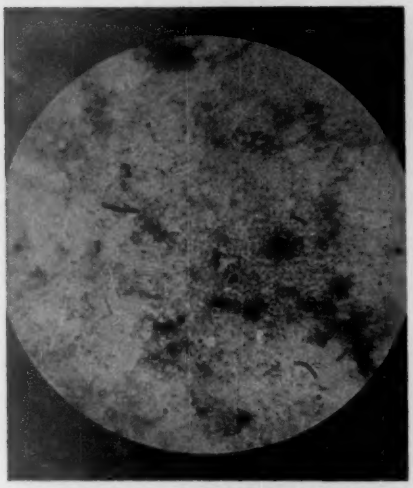

FIGURE 3.—Photomicrograph of anaerobic slime film after 15 days of growth in raw sewage (gram stain).

Rods were the predominant morphological group of bacteria present in slime growths on all "static slides." Both rods and cocci occurred as predominant groups at irregular intervals on "submerged dipping" slides, but rods were prominent during the first week. Rods and cocci were equally numerous on the first examination, but cocci were predominant thereafter on "alternate dipping" slides. Cocci were predominant on all "recirculation slides," except at 10 days, when rods were most abundant; this was the beginning of the period of most rapid increase in slime growth and cocci again became predominant at the peak and all through the sloughing period.

Anthophysa were found only on "static slides," and were largely predominant. They were numerous from the 8th to the 24th day, when mixed free-swimming ciliates and flagellates became most numerous, followed by a predominance of Vorticella on the 26th day. Vorticella were the predominant protozoa on almost all "submerged dipping" slides. Vorticella were initially most numerous on "alternate dipping" slides, but after 10 days Opercularia were dominant, except on the 22nd day when Carchesium appeared equally abundant. Opercularia was predominant on "recirculation slides" throughout the experiment, except on the 10th day when Euplotes, a free-swimming ciliate was most abundant. Miscellaneous free-swimming ciliates and flagellates occurred on most slides and amoeba were found on slides from all procedures except "static slides," being more frequent on "recirculation slides."

Nematodes were of scattered occurrence being found at least once on slides from each method except "submerged dipping." Rotifers were found on only two slides, both "recirculation slides."

Microscopical examinations indicate that there was not only a succession of organisms and a change in their relative numbers in building up the slime film, but also that there was a distinct difference in the predominant organisms under different conditions, even using the same sewage. The type of organism present under different conditions varied; but several forms were found to predominate, usually to the exclusion of a wide variety of forms.

It appears that the predominant type of bacteria and protozoa in slime growth may be a direct indication of the degree of aerobiosis or anaerobiosis. Cocci appear to be the predominant morphological group of bacteria under aerobic conditions while, rods appear more important anaerobically.

Attached protozoa were dominant on most slides from all methods, but the particular organism present varied. Anthophysa, the predominant protozoa on the anaerobic "static slides," was not found on slides from any other method. Opercularia was found only on the aerobic slides from "alternate dipping" and "recirculation," in which it was predominant. Vorticella was found most abundantly on the "submerged dipping" slides but was

of scattered occurrence on slides from all methods.

The slide film differs macroscopically as well as microscopically in appearance under differing degrees of motion and aeration.

Summary and Conclusions

The physical, chemical and biological composition of submerged slime growths have been described.

The moisture, ash, and nitrogen characteristics of the slime accumulations correspond to those of solids collected by settling of raw sewage. A total nitrogen content of one per cent is extremely low for a material of predominantly biological origin; it is concluded that, in addition to biological material, considerable amounts of sewage solids are imbedded in the slimes.

The biological composition of slimes developing in anaerobic and aerobic locations was the same during the primary slime formation period. The microfauna during this period consisted of a few flagellates and free-swimming ciliates. At the period of maximum slime formation, the content of slimes from anaerobic locations was predominantly flagellates. A more diversified fauna was present in slimes from aerobic locations (Vorticella, Carchesium, free-swimming ciliates and flagellates). Bacterial flora present in slimes from anaerobic and aerobic locations consisted of short and long rods and large coccoid cells. After the primary film formation period, Beggiotoa and Thiothrix appeared in the slimes both in aerobic and anaerobic location. As the film developed, zoogleal bacteria (gram positive cocci) became abundant.

Several different laboratory methods of submergence were studied, using microscope slides as the contact medium. The results of these studies demonstrated that slime formation involved a succession of organisms, a change in the relative number of organisms as the film developed, and a distinct difference in the predominant organisms with different methods. Attached protozoa were dominant on all slimes obtained by all methods of submergence, the type of attached protozoa varying with different methods of submergence. Antophysa was the predominant protozoa in anaerobic "static slides"; Opercularia in aerobic slides from "alternate dipping" and "recirculation slides;" and Vorticella in "submerged dipping slides."

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For further information write to Professor Emmett B. Moore, Chairman, Department of Civil Engineering, State College of Washington, Pullman, Wash.

COURT DECISIONS AND STATUTORY PROVISIONS IN WATER POLLUTION CONTROL LAW *

By Warren H. Resh

Assistant Attorney General, State of Wisconsin, Counsel for Wisconsin Committee on Water Pollution and Wisconsin State Board of Health

There is perhaps no area of social control more susceptible or subject to the exercise of the state's police power than that of water pollution. Polluted water is almost as bad as no water. In the light of the impact of an expanding industrial economy and tremendous population growth upon the purity of our public waters it is suggested that the old saying, "you never miss the water until the well runs dry," be amended and brought up to date by having it read: "you never miss the water until the well runs dry *or until it becomes polluted.*"

It has been said in one court decision that:

"The state may in the exercise of its police power require that sewage be purified before being cast into a stream, or it may absolutely prohibit the use of streams for such purpose. It has been stated that the right of a municipality to empty its sewage into a stream or a river is merely a legislative license revocable whenever public health and safety require. No constitutional property right is invaded by absolutely prohibiting the casting of any obnoxious matter into any running stream of the state. This power to regulate or prohibit may be delegated to the state board of health, and where its exercise is committed to the sound discretion of the board it may be exercised without taking evidence. . . ." 56 *Am. Jur.* "Waters" §388, pp. 813–14.

This discussion will rely to a greater extent upon cases from Wisconsin than upon cases from other jurisdictions, because of a somewhat greater familiarity of the author with the Wisconsin law. However, this discussion frequently will cite decisions from other jurisdictions.

State Power to Control Pollution

In *Winchell v. Waukesha,* (1901) 110 Wis. 101, 107, 85 N.W. 668, it was clearly held that the balancing of private rights to continued purity of lakes and streams against the discharge of sewage by a municipal corporation is entirely a question for the legislature.

Perhaps the most far-reaching case upholding the state's police power over water pollution is that of *Madison Metropolitan Sewage District v. Committee on Water Pollution,* (1951) 260 Wis. 229, 50 N.W. (2) 424. This case upheld a statute requiring the diversion of sewage plant effluent from the natural water course of the area, consisting of a series of shallow lakes where nuisance conditions resulted from the inorganic phosphorus and inorganic nitrogen contained in the effluent. The statute required either that the effluent be diverted to another water course on the theory that if discharged into a flowing stream nuisance conditions might be averted, or that in lieu thereof the community supply such advanced treatment of the effluent as would accomplish the same results as diversion. While the statute did not specifically name the city of Madison, it was really limited to that area be-

* Presented at 1955 Annual Meeting, Central States Sewage and Industrial Wastes Assn.; Rochester, Minn.; June 22–24, 1955.

cause of limiting factors such as effluent from a sewage treatment plant serving 45,000 persons or more being discharged into a lake of more than two square miles and less than six square miles in an area located within 10 miles of the treatment plant.

In this case the court rendered a sweeping decision holding that a municipal corporation has no privileges or immunities under the federal constitution which it may invoke against the state and that the authority of the legislature is supreme over a municipal corporation, subject only to such limitations as the constitution may prescribe. The court made it clear that the state in exercising its police power may require that sewage be purified before being cast into a stream, or it may prohibit the use of streams for such purpose entirely without violating any constitutional property rights. It was also said that a city is liable for the maintenance of a nuisance even when operating in its governmental capacity and that the question of classification for purposes of legislation is primarily for the legislature both as to need and basis; also that in matters involving only questions of public policy the determination of the legislature is final and conclusive on the courts.

Subsequent to that decision the sewerage district filed plans for diverting its effluent around the Madison lakes through Badfish Creek, the Yahara River and thence into the Rock River. Residents along these streams petitioned the Committee on Water Pollution to hold a hearing and disapprove of the project on the grounds that it would create a nuisance and irreparable damage to the riparian owners. After a 5-day hearing the committee made an order dismissing the complaint and refusing to disapprove of the diversion route. There is a bill in the legislature to repeal the statute in question and this has been amended to grant further time and provide for scientific research and investigation designed to perfect methods of eliminating the nutrient elements from the sewage effluent so as to make diversion unnecessary.

Factor of Nuisance Conditions

The foregoing is important because apparently this is the only instance in history where any state has gone so far as to provide by statute that even though sewage effluent is treated to the highest degree possible under modern sanitary engineering methods it still may not be discharged into the nearest water course if nuisance conditions result.

Most states have not gone this far either by way of regulatory legislation or by court decision, but there has been a considerable development in the case law relating to water pollution particularly insofar as the rights of individual property owners are concerned.

There are many cases which have been instituted either for injunctive relief to prohibit discharge of wastes into streams where nuisance conditions are alleged or to collect damages or both.

The legislative authority to construct and operate a sewage treatment plant does not permit the creation and maintenance of a nuisance. Generally speaking a sewage treatment plant is not a nuisance *per se*, although it can be such where improperly designed or improperly maintained and operated. Also it has been held that the right to pollute a stream by discharge of effluent from a treatment plant will not lead to a prescriptive right to continue such pollution; *Corsicana v. King*, (1928 Tex. Civ. App.) 3 S.W. 2d 857.

Balancing of Equities

The courts seem to be less ready to award injunctive relief than to award damages. The reasons many have refused injunctive relief are not always clear, but on the whole it would appear that the theory usually relied on is that there should be a balancing of

the equities. In other words where the abatement of the nuisance requires the discontinuance of the disposal system the comparative harm to the few that might be injured by its continuance should be weighed against the harm to the entire community which might be affected by its discontinuance. Sometimes also the courts indicate that equitable remedies such as injunctive relief are matters of discretion, which need not be allowed where there is adequate legal relief available through compensation and damages; *Fields Sewerage Co. v. Bishop,* (1930 Tex. Civ. App.) 30 S.W. 2d 412; and *Parsons v. City of Sioux Falls,* (1937) 65 S. D. 145, 272 N.W. 288.

Perhaps it should be added here that in the more recent cases the courts have been much more free in granting injunctions as well as damages than was true at an earlier stage of our history.

Pollution Cases Involving Sewage Treatment Plants

There are many cases involving the pollution of water courses by sewage treatment plants.

In *Harrisonville v. W. S. Dickey Clay Mfg. Co.,* (1933) 289 U. S. 334, where a sewage treatment plant removed only 60 per cent of the organic matter and it was shown that by further treatment an additional 30 per cent could be removed, the court considered that a nuisance resulted entitling the owner of pasturage land through which the stream flowed to compensatory damages equal to the loss of the land on account of continuance of the nuisance.

The case of *City of Lakeland v. State ex rel. Harris,* (1940) 143 Fla. 761, 197 So. 470, is an interesting nuisance case where the treatment plant was neither modern nor capable of purifying the matter which passed through it. The court held that the responsibility was always upon the municipality to refrain from maintaining a public nuisance, and it was no defense that the municipality was then unable to do so because of poverty.

In *Poovey v. Hickory,* (1936) 210 N. C. 630, 188 S.E. 78, a land owner was permitted to maintain an action against a municipality to abate a nuisance and recover damages resulting from the maintenance of an inadequate sewage treatment plant which permitted the discharge of raw sewage into a stream running through or over the plaintiff's land.

In *Cushing v. Luke,* (1921) 82 Okla. 189, 199 Pac. 578, a municipality was held liable for damages where its septic tank overflowed into a water course flowing over the plaintiff's land, seriously interfering with the comfort of the plaintiff in the occupancy, use and enjoyment of her home and premises.

In *Bethany v. Municipal Securities Co.,* (1954 Okla.) 274 Pac. 2d 363, recovery was permitted for the pollution of plaintiff's private lake in such a way as to destroy the fish with which it had been stocked. The evidence indicated that the defendant's sewage treatment plant was inadequate to handle the sewage from a town of the defendant's size and that an overflow of untreated sewage was discharged from the plant into the creek which was the only inlet to plaintiff's lake.

In *Veazey v. City of Durham,* (1950) 231 N. C. 357, 57 S.E. 2d 377, although a municipality had been granted an easement to transport sewage through a narrow strip of the plaintiff's farm through means of pipes, it was held that this did not bar an action predicated upon the allegation that the defendant had damaged the plaintiff's farm by casting sewage from its disposal plant into an open water course which traversed the farm.

The case of *Stovern v. Calmer,* (1929) 207 Ia. 1126, 224 N.W. 26, is an interesting one. After an injunction had been issued to restrain the municipality from committing a nuisance by discharging its raw sewage into a stream, the evidence showed that

the former nuisance had been abated by the erection of a sewage treatment plant so that the effluent had become colorless and only slightly flocculent, with the water in the stream having become clear. There was therefore no basis for continuance of the injunction.

Another case in which the court ruled that the city was not maintaining a nuisance is that of *People v. Reedley,* (1924) 66 Cal. App. 409, 226 Pac. 408, where the evidence showed that the city processed and treated the sewage before its discharge into the river rendering the sewage harmless, so that there was no probability of communication of disease.

Wisconsin Cases Involving Sewage or Industrial Wastes

A few Wisconsin cases, relating to both sewerage and industrial wastes, will be cited.

In *Winchell v. Waukesha, supra,* the city sewer system had an outlet into a navigable river about one-quarter mile above the plaintiff's farm. As the volume of sewage increased the defilement of the waters became apparent, and an extremely offensive, disagreeable odor pervaded the plaintiff's residence, causing great distress, discomfort and illness. The waters of the river were rendered unfit for bathing or for watering stock, and the volume of the water was only about double the amount of sewage.

Experts testified that at an expense of about $5,000 the sewage could be purified and rendered innoxious and inoffensive. It was held that under the circumstances there was an obvious nuisance, which if created and maintained by an individual would entitle the plaintiff to injunctive relief as well as damages.

The case of *Mitchell Realty Co. v. City of West Allis,* (1942) 184 Wis. 352, 199 N.W. 390, 35 A.L.R. 396, was an action against both a city and a private corporation contributing stream pollution and nuisance conditions. It was held that recovery could be had against all of the defendants for their appropriate share of the damage.

The case of *Middlestadt v. Waupaca Starch and Potato Co.,* (1896) 93 Wis. 1, 66 N.W. 713 involved the operation of a starch factory which polluted the waters of a natural stream and rendered them unfit for watering stock and ordinary domestic use. An injunction was granted.

Damages were awarded in the case of *Behnisch v. Cedarburg Dairy Co.,* (1923) 180 Wis. 34, 192 N.W. 447, where large quantities of skimmed milk were deposited in a stream which flowed through the plaintiff's land, resulting in unhealthful and deleterious odors which reduced the rental value of plaintiff's premises. The court here raised, but found it unnecessary to answer because of the factual situation, the question of whether or not a prescriptive right can be acquired to discharge wastes into a stream.

In the case of *State ex rel. Martin v. City of Juneau,* (1941) 238 Wis. 564, 300 N.W. 187, it was held that the discretion vested in the state board of health and in the state committee on water pollution by the statute, empowering those bodies to make orders relating to the control of surface waters, was not arbitrary in view of statutory provisions making such discretion subject to court review. It was also held that the power of these agencies to control water pollution extends to prevention as well as to the limiting of conditions which are destructive of the public health, and that neither the board nor the committee need postpone action until the health of the community is impaired or some citizen has died as a result of the pollution of the water.

Obligation of Municipality to Treat Industrial Wastes

An interesting question in any comprehensive study of water pollution

control is that of whether a municipality is obligated to construct a sewage treatment plant which will successfully handle any and all types of industrial wastes.

According to *McQuillen Municipal Corporations,* Second Edition Revised, Vol. 4, §1552, the prevailing rule is that the municipality is the sole judge of the necessity or desirability of sewers and drains in the absence of any mandatory legislation.

Ordinarily the construction or repair of sewers and drains involves the exercise of discretionary powers which the courts undertake to control; 38 *Am. Jur.* "Municipal Corporations," §565, p. 253.

In *Hart v. Neillsville,* (1905) 125 Wis. 546, 104 N.W. 699, it was held that a city having by its governing body duly adopted a plan for a sewage system and executed the same it is not liable for injuries caused thereby to private property not involving an unconstitutional taking thereof, produced by defects in such plan. Hence it would appear that industries could not mandamus the city to adopt a type of sewage treatment to handle their industrial wastes, nor recover damages for the failure of the city to construct a type of plant capable of handling particular industrial wastes.

In *Benson v. City of Andalusia,* (Ala. 1940) 195 So. 443, the court held that the right of a municipality to regulate and control the use of its sewer system was a necessary incident to its ownership; and in *State ex rel. F. B. Co. v. Village of Beachwood,* (Ohio 1942) 46 N.E. 2d 808, it was held that a municipality has the power to fix reasonable conditions for the use of its sewers and water mains.

Joint Polluters

The case of *Thorne v. Houcut Dredging Co.,* (1941) 43 Cal. App. (2) 737, 111 Pac. (2) 368, is another case like that of the *Mitchell Realty Co. v. City of West Allis,* (1924) 184 Wis. 352, 199 N.W. 390, 35 A.L.R. 396, in Wisconsin where pollution resulted from several sources. It was held that one who wrongfully polluted a stream by conducting a mining enterprise thereon was responsible only for the actual damages resulting from his own acts to the riparian rights of others, and that he could not be charged with damages resulting from other persons' wrongful pollution of streams independently of his acts.

Another case of joint liability was that of *Jessup and Moore Paper Co. v. Zeitler,* (1942) 180 Md. 395, 24 A. (2) 788. This case takes a somewhat different approach than the preceding one where each polluter was held liable for his own acts only. In the Maryland case it was held that two or more persons polluting a stream are jointly and severally liable for the damages so that an injured person may recover against one or all contributing to the injury. It was stated that it is no defense to allege that other persons are committing similar acts. The court indicated, however, that each wrongdoer would be liable in a separate action and would be subject to injunction.

Damages and Proof

How great pollution must be to entitle one to damages is a difficult problem. In the case of *Ravndal v. Northfork Placers,* (1939) 60 Idaho 305, 91 Pac. (2) 368, it was held that a farmer could not complain of a reasonable use of water by a miner higher up stream although such use pollutes and makes the water less desirable. The court indicated that there must be substantial injury to warrant relief.

Another case involving a different type of pollution was that of *Michelsen v. Leskowicz,* (1945) 270 App. Div. 1042, 55 N.Y.S. (2) 831, 63 N.Y.S. (2) 191. It was held there that where a duck farm business had long been maintained by the defendants, the plaintiffs by selecting adjoining riparian property for a country home were

not entitled to have the farm declared a nuisance simply because the air was not altogether free from either the quacking of ducks or the occasional odor therefrom or because the stream contained fecal matter. The court seemed to indicate that a duck farmer had the same right to make use of a stream as would a livestock farmer.

The case of *Klassen v. Central Kansas Cooperative Creamery Assn.*, (1946) 160 Kan. 697, 165 Pac. (2) 601, is an interesting one. There a creamery association permitted harmful waste products to injure a farm tenant by polluting a stream and underground water supply as a result of the deposit of excessive quantities of wastes in the city sewer system. The court held that the company was liable to the tenant for damages, regardless of whether pollution was a result of the defendant's negligence or improper conduct. The significant thing to note is that the liability of the defendant was in no wise affected by the fact that there was an intervening municipal sewer system.

The question of the degree of proof required is always a troublesome one in any lawsuit for damages. In *Masonite Corp. v. Steele,* (1945) 198 Miss. 530, 23 So. (2) 463 and 756, it was held that a resort proprietor who claimed that the patrons of his fishing resort were reduced by the killing of fish by the wood fiber effluent of the defendant must prove that his profits were less than they would have been had the fish not been killed.

In *Sutherland Co. v. Aaron,* (1954 Miss.) 72 So. (2) 161, it was stated that the measure of damages, where there is no permanent injury to land if the pollution were discontinued, is the depreciation in rental value; but where the damage is permanent, the measure of damages is the difference between the market value of the property before and after the creation of the nuisance.

Powers of Water Pollution Control Agencies

With the growth and development of statutory law setting up water pollution control agencies, the question of possible conflicts between the powers of such agencies and the powers of the courts occasionally arise.

In the case of *Satren v. Hader Cooperative Cheese Factory,* (1938) 202 Minn. 553, 279 N.W. 361, it was held that a statute giving the state board of health the power to administer and enforce pollution laws did not deprive the district court of its power to issue an injunction to abate a private nuisance arising from the discharge of whey into a creek, notwithstanding that the damage to the plaintiff's farm was only $63 a year. The court rejected the argument that an injunction should not be issued where injury is insignificant and where it would prevent the operation of a large business of great value to the community.

The same question was involved in the case of *Barrington Hills Country Club v. Village of Barrington,* (1934) 357 Ill. 11, 191 N.E. 239, where it was stated that as against riparian owners a village has no inherent right to use a creek as an outlet for its sewage system, even though the creek is the only natural water course in the area and also notwithstanding that some pollution existed from other sources. It was held not to be a proper case for balancing equities, and that even though the treatment plant was approved and authorized by the sanitary water board this would not authorize encroachment upon the private property rights of riparian owners, nor bar a suit in equity to enjoin a private nuisance. It might be noted in passing that the treatment plant here was equipped with an Imhoff tank and that storm waters were not separated and that there was also considerable industrial waste from creameries, cheese factories and so forth. Whether the court would

have granted relief in this case, had there been anything like adequate treatment facilities, of course would present an entirely different question.

The case of *City of Huntington v. State Water Commission,* (1953 W. Va.) 73 S.E. (2) 833 is an interesting one in that a city proceeded against the order of the state water commission requiring it to cease polluting certain waters, and advanced the contention that the city did not have sufficient unused revenue to comply with the order, and that the value of taxable property was not sufficient to authorize bonded indebtedness to comply with the order of the commission. The court said that there must first be a good faith effort to raise the necessary funds to comply with the order and that public health is a matter of state-wide concern in which the power of the state is supreme.

Other Cases Cited

Among other cases involving the authority of state water control agencies are the following:

Sanitary Water Board v. Anthony, (1955 Pa. Com. Pl.) 66 Dauph. 250. This involved the Pennsylvania Clean Streams Act of 1945, 35 P.S. §691.1 et seq. which divides the waters of the commonwealth into three categories:

1. Clean waters which are devoted to a public use.
2. Clean waters.
3. Polluted or unclean waters.

The court stated that it is the policy of the state to protect waters falling in the first class and that it is a nuisance to pollute a stream from which the public gets its water supply. The court sustained an order of the sanitary water board denying permission to conduct strip mining operations because of the possibility of acid mine drainage.

State Water Commission v. City of Norwich, (1955) 141 Conn. 442, 107 A. (2) 270 involved an order of the state water commission directing the city to take steps to construct a sewage treatment plant, and the city failed within the time limited by statute to appeal on the question of the reasonableness of the order. It was held that the city could not complain about the reasonableness of the order on appeal from a judgment of the superior court enforcing the order of the commission.

The last case to be cited is a Wisconsin case involving somewhat the same principle. In *City of Superior v. Committee on Water Pollution,* (1953), 263 Wis. 23, 56 N.W. (2) 501, it was held that a city, whether challenging a sewage treatment order of the state committee on water pollution on the grounds of excess of statutory authority exercised by the committee, or on constitutional grounds or on some other ground, could obtain a judicial review of the order only by following the method provided in the statute under the uniform administrative procedure act; and that the city could not in lieu thereof obtain a review of the order through the medium of an action for declaratory judgment. In other words, where a specified method of review is prescribed by statute the method so prescribed is exclusive. This is important, for the reason that municipalities and industries which are delinquent in complying with orders of water pollution control agencies frequently are also delinquent in seeking review of the orders in the courts in the manner and within the time prescribed by statutes, and the doctrine of the cases seems to be that they are then left without any defense to the enforcement of such an order.

Summary

It would be most misleading to intimate that the cases just discussed embody all or even a minute fraction of the many decisions throughout the country on water pollution. Rather, these cases have been selected at ran-

dom to illustrate various phases of the problems that have troubled the courts and the administrative agencies charged with the enforcement of water pollution control statutes.

It may be stated, in general, that the more recent decisions and the more recent legislation portray an increasing awareness and sensitivity to the problem by the courts as well as by the various state legislatures. The latter have set up more and more water pollution control agencies with varying powers.

It is a far cry from the old days when powerful industrial interests controlled the legislatures, if not the courts, in some of the states; and when, in some instances, pollution was permitted to nearly destroy one of our greatest economic resources, our public waters. Economic self-interest on the part of industries, which must have clean waters in order to survive, as well as articulate demands from conservation and other groups interested in preserving the purity of our lakes and streams for recreational use have dictated the change in governmental philosophy.

DISCUSSION

By Clarence W. Klassen

Chief Sanitary Engineer, Illinois Department of Public Health, and Technical Secretary, Illinois Sanitary Water Board, Springfield, Ill.

Mr. Resh has approached legal questions involved in water pollution control law from a somewhat different standpoint than is normally presented. Not only were specific court cases cited, but they were discussed under several headings which make this paper more valuable for ready reference to legal decisions. As an example, the first heading was "State Power to Control Pollution."

Mr. Resh stresses that the area of pollution control is particularly susceptible to exercise of the state's police power. As an administrator of a state pollution control program, the author of this discussion has always appreciated the need for the legal control but considers that every legal case involving stream pollution indicates the failure on the part of someone, and very often the regulatory agency, to work out a voluntary solution. In other words, the legal aspects of stream pollution present the negative side of the pollution picture. Cases of individual action against an alleged polluter, however, often fall into an entirely different category than some cases presented by the author of this fine paper. In some instances, it is the sheer desire to collect money, and in other instances the abatement of pollution is the principal motive. It is appreciated, however, that legal action often is necessary and may be a means to an end in securing abatement.

The author recognized the fact that water pollution control is a part of an over-all water resources conservation program and that, while industry has a responsibility for taking care of its wastes, industry likewise has a big stake in this important natural resource—water.

The point is also stressed that municipalities and industries alike have the right to use a stream but not the right to abuse it; and that it is definitely within the power of a state to control that particular use from the standpoint of discharging obnoxious or polluting material into the public waters.

In Illinois, the public waters have been looked upon as public property, and it is part of the job of the state stream pollution control agency to de-

termine for the public how their property is being used or possibly misused.

Mr. Resh points out that a state legislature is supreme over a municipal corporation, subject only to such limitations as the constitution of the state may prescribe. However, it should also be pointed out that a state stream pollution control body cannot take away the right of pure water from the riparian owners, for example, through the classification of certain streams to set them aside as waters that could be of inferior quality.

Of particular interest is the statement that the courts seemed to be less ready to award injunctive relief than to award damages. This is a fact, of course, but recently in Illinois the court not only granted the State Sanitary Water Board an injunction against an industrial corporation, but also, following this, when the corporation caused stream pollution the court subsequently found the corporation in contempt of court and fined it. This was the case of the *State of Illinois* v. *National Petro-Chemicals Corporation*. The judge in ruling on this stated that while he recognized the value economically to the community of this large corporation it did not have the right to in any way make the water in the outlet stream detrimental to the use of farmers and other riparian owners downstream.

Legal examples cited by Mr. Resh under the heading of "Pollution Cases Involving Sewage Treatment Plants" should be extremely interesting to all now operating sewage treatment works. Many treatment plants are becoming obsolete or outgrown, and some municipalities already are approaching the day when a second generation of sewage treatment plants is needed. It has been interesting to note from examples cited that very often the same type of litigation is needed to secure sewage treatment plant additions and improvements as was involved in the original action to secure a plant to treat sewage which was being discharged raw to the outlet stream.

Of particular interest to every municipality are citations under the heading "Obligation of Municipality to Treat Industrial Wastes." In Illinois, for example, when a municipality accepts the connection of an industrial plant into its sewer system it likewise accepts the responsibility for the treatment of such wastes. The Illinois laws, which are similar to those of most states, give the cities and villages the right to regulate the use of their particular sewer systems.

From the industrial viewpoint, the case of *Hart* v. *Neillsville* is particularly interesting: it outlines definitely the responsibility of the industry for taking care of its own wastes, and merely because the industry is located in a municipality it does not follow that the town is obligated to treat the industry's wastes.

It might be of interest to note that some recent court decisions in Illinois recognized the fact that, in an incorporated municipality, an individual could own and control a sewer located in a public thoroughfare. This situation could materially complicate the legal picture; for example, if pollution was caused by that particular sewer outlet. As yet no court case has involved such a complicated procedure.

Under "Damages and Proof" some excellent examples are cited for reference. Experience in Illinois has demonstrated that it is extremely difficult to prove damages; for example, veterinarians, physicians or other scientific personnel very often will give a "curbstone opinion" to a person who might be the complainant, but when it comes to appearing on the witness stand and making statements under oath it is often quite a different story.

The Illinois case cited of *Barrington Hills Country Club* v. *Village of Barington* is an example of the result of legal action which may result in what

engineers and sewage treatment operators would consider a quite impractical situation. In this case, among other decisions, the judge ruled that no municipality could discharge either treated or untreated sewage into any stream in Illinois. While this decision still stands it obviously is so impractical that it has been completely disregarded; it would, of course, necessitate reversion to septic tanks or privies and to the disposal of effluent on each individual property.

The author of this discussion was pleased to have Mr. Resh include some legal discussion of whether or not a state can decide that some waters can be or cannot be polluted. At times it is claimed that sewage treatment is required merely for the sake of treatment rather than because of any demonstrable pollution. This is a somewhat controversial subject, but undoubtedly there are instances where certain indsutrial wastes would not require treatment prior to being discharged into a stream or a surface body of water. This, of course, recognizes the ability of water to absorb a certain pollutional load.

As an engineer, it was particularly gratifying to review the paper by Mr. Resh and to note his practical views and comments on this aspect of pollution. This discussor wishes to compliment the author, and also the State of Wisconsin for having on its legal staff a person who can present the excellent legal approach and comments which were evidenced in this paper.

CRITICAL OCCUPATIONS

Professional engineers in all branches, engineer draftsmen on design work, and operators of water and sewerage facilities are included in the most recent listing of critical occupations released by the Office of Defense Mobilization. The list is used by the Selective Service System as a guide in selecting men with critical civilian skills for enlistment in the Ready Reserve under Executive Order 10650, signed by the President, January 6, 1956.

Young men possessing the listed skills and employed in the designated activities who are selected by the Selective Service System under the Executive Order will be eligible to complete their active duty for training in from three to six months, thus minimizing the time lost from critical civilian occupations. They will perform the balance of their military obligation in the Ready Reserve. Ordinarily they would be required to perform active military service for two or more years, plus service in the Ready Reserve.

THE OPERATOR'S CORNER

CONDUCTED BY DONALD P. SCHIESSWOHL

COUNTERACTION OF SEWAGE ODORS *

BY NICHOLAS POST

Airkem, Inc., New York, N.Y.

In the practical field of air pollution abatement in the past, the destruction of odors has often had the aspect of a hopeless task. The field of physical and chemical destruction of odors has been explored endlessly. Mechanical equipment for this purpose has proved to be inadequate and costly. And, while practically every known combination of chemicals has been tried at one time or another in an attempt to destroy odors by chemical reaction, most chemicals that are known to be effective as powerful deodorants against air-borne odors are toxic to human beings in the concentrations required for odor destruction in the atmosphere.

The treatment of sewage and industrial wastes has become an extremely important aspect of modern living and the designing engineers and operators of modern sewage treatment stations are to be commended on the excellent work they have done in attempting to control odor nuisances. There are still odor-control problems to be solved, however, in the handling of sewage for, while treatment and disposal are conducted under careful bacteriological control and advanced engineering design, some plants are subjected to the most complex and obnoxious mixtures conceivable. The liquid wastes from industrial activity alone are seemingly endless in variety. These wastes come from dye works, chemical plants, synthetic rubber plants, oil wells, refineries, gas plants, milk condenseries, paper mills, slaughter houses, glue factories, woolen mills, breweries, food products plants, and many other industrial establishments.

In wastes from most of these sources there is adequate organic matter of nitrogenous or sulfurous nature to give rise to the noxious products of biological decomposition. From the standpoint of odor, the worst of these gases are sulfides, mercaptans, indoles and amines. In addition to organic products, the waste waters from many establishments contain dissolved gases of malodorous quality that are little affected by the natural forces of self-purification in the water course or lake. These not only affect the water supply but may also contaminate the air for miles to degrade the surrounding commercial shorelines, bathing beaches and resorts. Many odorous industrial plants are removing odorous effluents from their stacks by the use of cyclone washers or scrubbers and chemical reaction. Unfortunately, a good many of these wastes are not readily oxidized or consumed as organic food, and treatment for odor by use of chemicals may create new odor problems.

* Presented at 1955 Annual Meeting, Virginia Industrial Wastes and Sewage Works Assn.; Alexandria, Va.; May 5–6, 1955.

Odor Perception

Since hydrogen sulfide, methyl mercaptans and methyl amines are all odorous offenders, one might conclude that the elimination of hydrogen sulfide would be the first line of attack. It is well known that hydrogen sulfide is a very toxic material and there have been reports that some sewage treatment plants have dispersed sufficient quantities to cause physical damage to plant equipment. A condition of this nature, of course, should be corrected at once. In sewage odor perception, however, hydrogen sulfide is not always the primary problem.

In the field of odor perception there are four facts of primary importance which must be taken into consideration, even though they cannot be demonstrated by chemical analysis. These are as follows:

1. Weak odors are not perceived in the presence of strong ones.
2. Odors of the same strength sometimes blend to produce a combination in which one or both of the components is unrecognizable.
3. In the presence of odors of constant intensity we rather quickly lose awareness of any odor at all.
4. Our like and dislike for any odor depends largely on association of the scent with pleasant or unpleasant experiences and impressions in the past.

Odor appraisals by the author of some 300 sewage plants in the Midwest, the Gulf Region, the Atlantic Coast and the West Coast indicated that hydrogen sulfide is not the typical sewage odor characteristic. The similarity of sewage odors was more noteworthy than any dissimilarity, in spite of variations in hydrogen sulfide. It was therefore concluded that combinations of methyl mercaptans, methyl sulfides and amines, in addition to indoles and skatoles, are the major offenders in sewage odor perception. When this combination of odors is in the air stream, there occurs what may be termed the "typical sewage odor." While this odor characteristic in all plants visited was very similar, it varied in intensity. Local conditions and types of treatment may increase or decrease any of the aforementioned odor combinations.

Odor Control Techniques

Methods now commercially available for control of odors by treatment of odorous air may be divided into four general classifications: combustion, absorption, use of odor modification, and counteraction. The first three methods, with their operating problems and limitations, have been described elsewhere in the literature, notably by Weaver (1). The author is primarily concerned with describing the method of counteraction, previously treated by von Bergen (2).

Principle of Counteraction

A very limited number of scientists have directed their attention to the field of odor perception. Around the turn of the century, Hendrick Zwaardemaker, a Flemish scientist, discovered that two substances, odorous singly, may be inodorous together. Using a double olfactometer, he experimented with many pairs of odorous substances, introducing them individually to either nostril to measure the ratio of quantities required for neutralization or counteraction. By this method, all possibility of chemical reaction between the two substances was removed. Some of the recorded counteracting odors were cedarwood *vs.* rubber; rubber *vs.* wax; wax *vs.* balsam of Tolu; and paraffin *vs.* wax. It is important to note that the true neutralization or counteraction of odors leaves no overriding odor, pleasant or unpleasant. There is some confusion in both the technical and popular press regarding the terms "odor masking" and "odor modification" or "odor

counteraction." Strong odors tend to mask weaker ones. If two odors are of about equal strength, a blend of the two is perceived or possibly both are identified. If one odor is considerably stronger than the other, as a rule it alone is perceived. This is masking. On the other hand certain pairs of odors, in appropriate relative concentrations, are antagonistic; and, when the two are sniffed together, the intensity of each odor is diminished. This is counteraction. The field of odor control by counteraction, as developed today, is based on the significant work done by Zwaardemaker. The development of odor counteractants has been a matter of pure research and many years have been spent in this delicate work.

There is no widely accepted classification of odors and, as a matter of fact, the description of odors usually depends on comparison. Sewage odors are complex groups of odors. It is unusual to obtain a specific odor from a sewage treatment plant and, even when this is the case, the odor may combine with transient odors in the surrounding atmosphere. For this reason waste odor counteractant formulations, although designed for specific odor descriptions, may contain complex groups of odor counteractants. There is no chemical method known for determining the effectiveness of odor counteractants. They are, however, successful in odor perceptive work and, in the last analysis, odor perception trials are the criteria for the effectiveness of any odor abatement method.

The Application of Odor Counteractants

The principle of odor counteraction deals with the molecules of malodor. In the process of odor perception the olfactory tract is so located that ordinarily only gas reaches it. Gas, of course, includes vapor. The sensitive areas are at the closed ends of channels that are readily penetrated only by diffusion, and the diffusion of solid and liquid particles of more than submicroscopic size is almost negligible. Accordingly, characteristic odors are rarely, if ever, produced by a smoke or mist of solid or liquid particles, however fine, except in the case of materials volatile enough to be smelled when only the vapor comes from massive forms of them. For practical purposes an odor is always a gas molecularly dispersed in the air in sufficient concentration to be above the threshold level of perception. For this reason, odor counteractants are most effective when vaporized and combined with the air stream by molecular dispersion. Regardless of whether the stream of odors is held to the ground by air inversion or is vertically or horizontally dispersed, the counteractant is carried with the malodor and is noticeably effective until the odor is dispersed beyond the range of perceptibility.

There are many sources of odor within the sewage treatment plant. Open grit handling areas, bar racks and screens, sedimentation areas, drying beds, sewage vents, manholes and lift stations may all become odorous. The odors from these sources expand by molecular dispersion in a rough range of five to one until the odorous pool is taken over by air movement. Further dispersal, mixing and dilution takes place during the variable air movements until the characteristic odor of sewage is perceived in the complaining area.

Odor counteractants are vaporized in much the same way by atomizing the counteractant into the air stream by means of a calibrated atomizing nozzle. These vaporizing points are usually located at the ground level, on the roof of a plant structure, and in the vicinity of the most odorous effluents. This provides vertical coverage from the bottom to the top of the odorous pool. The odor counteractants mix with the odorous air stream by

molecular dispersion and air movements, and are designed to follow the physical behavior pattern of the odorous elements. Some odorous elements disperse beyond the limits of perceptibility rapidly; others are more tenacious and do not disperse as quickly, either horizontally or vertically. Many groups of odors continue to travel and vaporize for miles. Specific odor counteractants are designed to follow these physical dispersal patterns.

Vaporization of the liquid odor counteractant is accomplished by clean, compressed air (approximately 2 c.f.m. per gun). The liquid odor counteractant is stored in a drum in a convenient location and fed through small connecting lines to the vaporizing nozzles by air pressure. The operation is automatic and, once the installation is set up and balanced, requires little maintenance except for weekly filling of the containers. Odor sources, vaporizing at a continuous rate, are treated continuously. The atomizing equipment is simple, requires little or no service and is readily adaptable to any area. It is easy to install, no alterations are required in the process equipment, and no downtime is required for installation.

In one activated sludge plant with an average daily sewage flow of 12 m.g.d., two guns were mounted to vaporize in the area of the aeration tanks. Another gun was located at the grit removal chamber to take care of the odors from this point and also those odors which might originate from the sludge drying beds nearby. A specific odor counteractant was used for these odorous components.

In a second treatment plant which treats the wastes from a population of some 20,000 people plus the industrial wastes from a pharmaceutical laboratory, two guns were installed at the trickling filters to operate automatically. At this plant the odor is most noticeable during the late afternoon and early evening hours so counteraction is placed in operation only during these hours when the odors are most severe.

In a third treatment plant hydrogen sulfide escaping from a raw sewage inlet structure caused numerous complaints. A vaporizing gun was installed at the top of this inlet for atmospheric treatment. The enclosed area within the structure was not treated. It should be pointed out, however that, without its warning characteristic odor, hydrogen sulfide can be very deadly. It should never be treated for odor control until it has been discharged at a point where dispersal is sufficient to reduce the parts per million below the maximum permissible limits for toxicity.

Dispersion of Odors

The atmosphere surrounding a sewage treatment plant usually has great capacity to disperse odorous effluents. Under favorable weather conditions the odors from a plant will rise gradually as they flow downwind and the contaminants will be dispersed until only a negligible concentration prevails in the atmosphere. Under these conditions and under conditions of actual vertical dispersion, odors do not ordinarily become an annoyance in the surrounding area. Unfortunately, there are many natural influences which arise to disturb this orderly dispersion. These adverse influences are aerodynamic and meteorologic. Wind flowing past a treatment plant generates a turbulence in the wake of the digesters and of the buildings. Turbulent masses of odor above and behind the plant are brought down to the ground in a spreading area. This is termed "downwash." Under these circumstances concentrations of odors may be very high in an area close to the plant. If the odor escapes the downwash situation it may flow downwind and come under the adverse influence of terrain. Hills, valleys, bays and pockets may set up currents which

entrap the odors. Thermal influences in the atmosphere play a large part in the over-all air movement. Under conditions of air inversion large quantities of odors may be brought to the ground over a large area and sometimes held there for hours. The location of the treatment plant is also of great importance. It may be located in a deep valley or on level ground, in sparsely settled territory or a community where there are many obstructions to smooth air flow. These factors all affect the dispersion and the diffusion of odors, the reaction of the surrounding community to them, and are important to valid odor survey work.

The odor counteraction method of application is adaptable to favorable and unfavorable air movements. Under conditions of good vertical dispersion or rising horizontal dispersions the application can be cut off. Under conditions of air inversion or wind velocities that create building turbulence and thermal looping, the counteractant may be dispersed into the air stream. Many sewage treatment plants are located in areas where the prevailing wind may be away from the populated or residential areas. To meet such conditions, wind control devices have been designed that automatically apply the odor counteractant when the wind is in the direction of the complaining area and stop the application when the wind direction is towards an unpopulated, or non-complaining, area.

Public opinion is an important factor in air pollution control and lack of an understanding public relations attitude towards the problem of air pollution is a serious matter. Good relations with the general public are extremely important for an aroused public is an unreasonable public, and an unreasonable public can cause excessive expenditures. It is, therefore, essential that industries and municipalities survey their positions to determine the most advantageous course of action in reference to air pollution control.

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GATE VALVE DESIGN AND APPLICATION *

By G. J. Manahan

General Sales Manager, The A. P. Smith Manufacturing Co., East Orange, N. J.

The gate valve is one of the oldest practical devices known and the one most frequently employed to control the flow of fluids and gases in pressure conduits. The term "valve" is now applied to any device that opens and closes and provides a passageway for fluids and gases.

Historical Background

The earliest form of mechanical valve was the bellows which was used by the Egyptians 3,500 years ago. Paintings on the walls of a tomb of that period picture a bellows with bamboo piping used to smelt ore. The next development was the flap type valve which allowed fluids to travel in one direction in open and low-head conduits. Grecian illustrations dating about 600 B.C. depict showers used for bathing which involved the discharge of water controlled by flap type valves. About 250 B.C., a Greek developed a two-cylinder pump which employed a valve in connection with the functioning of each cylinder.

* Presented at the 1955 Annual Meeting, Virginia Industrial Wastes and Sewage Works Assn.; Alexandria, Va.; May 5–6, 1955.

A century or more before the Christian era, Hero of Alexandria is credited with the development of the first workable hinged valve involving a flat gate which was opened and closed to admit water and control the discharge of pumps. In his writings, Hero wisely stipulated that the seat facings should be smooth and polished to insure a reasonably tight closure.

The fourteenth century thinker, artist and engineer, Leonardo daVinci, left many drawings of valve constructions, the most advanced of which was the multiport check valve which involved four pairs of hinged gates. Today multiport valves find a ready use in water and sewage facilities.

Prior to the eighteenth century, pressure conduits were not generally employed because a satisfactory piping material was not available. Early in the eighteenth century, the first cast iron pipe was installed and in 1795, Joseph Bramah designed a valve stem featuring a screw thread which was used to open and close sluice type valves.

In 1839 James Nasmyth designed the first practical metal gate valve which was immediately employed by the East London (Eng.) Water Works. The Nasmyth design consisted of a cast iron body and bonnet, a threaded valve stem and a solid tapered plug.

Solid wedge, taper seat valves were introduced in the United States a few years later. About 1845 existing water utilities started to employ gate valves in pressure piping. Operating pressures were moderate, pipe sizes were reasonably small, and most of the valve development work related to small diameter and low pressure valves. The major changes and refinements of the original Nasmyth solid wedge, taper seat valve took place between 1845 and 1895. During this period, double disc, parallel and taper seat gate valves were developed and met with wide and continuing use. The first double disc, parallel seat gate valve was of the bottom wedging type and was followed by center and side wedging types.

Gate Valve Types

The solid wedge, taper seat gate valve design has not been changed basically during the past 110 years. In this design, it is essential that the taper of the plug must coincide exactly with the taper of the body seats to achieve a pressure seal or shut-off. To prevent the plug from being moved in the direction of the downstream seat by the flow during the operating cycle, and to eliminate valve stem strain and undue seat face wear, plug guides should be integral with the valve body. The guides should have non-ferrous metal facings. As valve sizes increased, it was found that the problem of maintaining a perfect mating of the plug taper and the body seat taper was diffi-

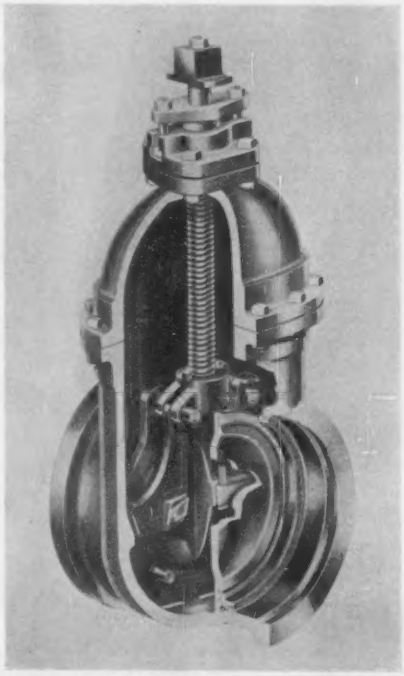

FIGURE 1.—Exposed view of a double disc, side wedge, parallel seat, non-rising stem gate valve.

cult and expensive to achieve. It was also noted that the body seat taper was distorted as operating pressures and dead weight piping loads transmitted to the valves increased, with the result that leakage took place.

The Coffin double disc taper seat gate valve was developed in 1867 to obviate and minimize the distortion of the body seat and the resulting leakage.

Double Disc, Parallel Seat, Bottom Wedging

S. J. Peet, in 1875, patented the double disc, parallel seat, bottom wedging type gate valve (Figures 1 and 2). In this design, the discs move downward until the lower wedge travel is terminated by contact with a stop in the bottom of the valve body. The discs and upper wedge continue to travel until the wedges contact one another, thus forcing the discs against the body seats. In opening, the first turn or initial movement of the valve stem releases the wedges and allows the valve discs to travel freely to the open position. Suspended solids which are sometimes present in the bottom of piping can be entrapped by the bottom wedge, changing the contact elevation, thus occasioning preliminary or early wedging with the discs in a partially open position.

Double Disc, Parallel Seat, Center and Side Wedging

Center and side wedging, double disc, parallel seat type gate valves were developed and introduced late in the nineteenth century. In the side wedging design, the discs move downward until the travel is terminated by contact with stops in the bottom of the valve body, at which point the disc faces fully cover but are not in firm contact with the body seat ring faces. The final rotation or movement of the valve stem actuates the wedging mechanism expanding and forcing the disc faces against, and in contact with, the body seat faces, producing a drop-tight shut-off. The wedging mechanism is actuated by inclines integral with the valve body located near the horizontal centerline of the valve; thus, suspended solids and foreign matter in the bottom of the piping cannot occasion preliminary or early wedging. In opening, the first turn or the initial movement of the valve stem releases the wedges before the discs move, thus the discs are free to travel to the open position without undue surface friction.

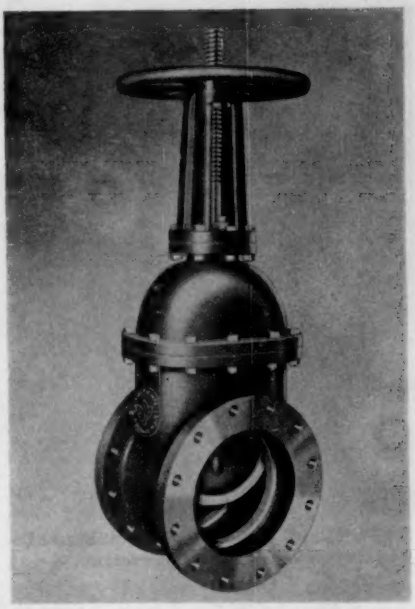

FIGURE 2.—Flanged (O.S.G.Y.) rising stem, double disc, side wedge, parallel seat gate valve.

Square Bottom Case and Disc

Manual, cylinder and electric motor operated gate valves which will be subject to frequent operation and in service will be either in a fully open or a fully closed position should be of the square bottom case and disc type. This construction prevents the downstream disc from tilting into the downstream port opening. The disc and disc rings, at the bottom, are essentially square with the result that a substantial area

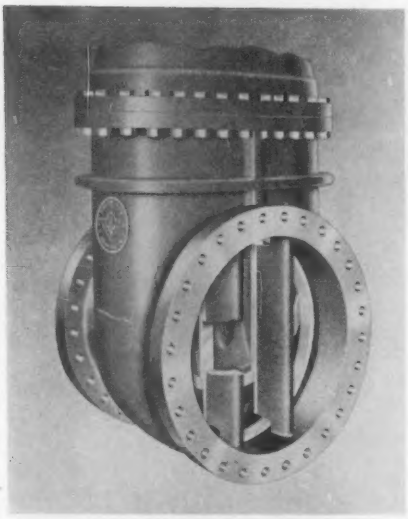

FIGURE 3.—Exposed view of beamed waterway valve construction.

of each disc ring is in contact with each body seat ring during the operating cycle. Disc ring and seat ring wear is thereby reduced to the practical minimum.

Valves installed in a flat position in vertical piping should be of the square bottom case and disc construction with bronze tracks secured in the top of the valve body and in the bonnet, providing parallel surfaces supporting the discs during the operating cycle.

Beamed Waterway

Valves which will be subject to frequent operation, substantial unbalanced pressure, high velocity and will be used to control flow should be either beamed waterway (Figure 3) or single throttle construction if the flow is not subject to reversal. If the flow is subject to reversal, the valves should be double throttle construction.

The beamed waterway valve design prevents the downstream disc from tilting into the downstream port opening. Bronze faced beams located on the downstream side of the valve body register with bronze facings securely attached to the downstream disc. The beam faces and the disc facings are in the same plane as the disc and seat ring faces. The beams prevent disc tilting and in combination with increased contact bearing area, uneven disc and seat ring wear is eliminated.

Throttle Construction

Throttle construction mechanically lifts each disc ring face out of contact with each seat ring face during the entire travel of the discs. This construction (Figure 4) stops disc tilting, uneven disc and seat ring wear and chatter. Throttle construction valves are suitable for installation in vertical, horizontal (on edge) or in flat positions in vertical pipelines. The disc and seat ring faces are not in contact while the valve is in a partially open position or while the discs are moving.

Throttle construction discs are provided with three non-ferrous metal shoes. One shoe is attached to the disc at the top centerline, the other shoes are attached to lugs which are integral

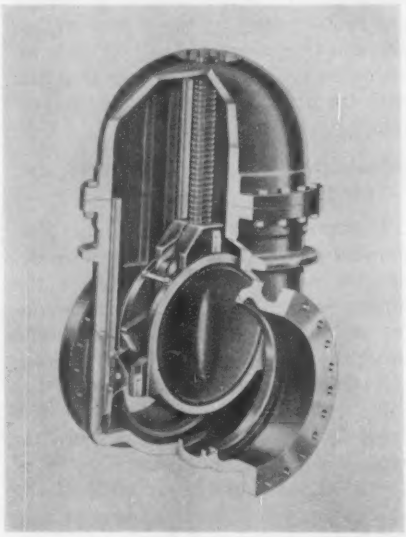

FIGURE 4.—Exposed view of throttle construction gate valve with disc shoes and body tracks.

with the disc, one on each side near the bottom of the disc.

The throttle construction valve bodies have three non-ferrous metal tracks. Two tracks are located and securely attached to the body, one on each side of the body seat ring. The third track is located and securely attached at the top centerline above the seat ring. This track extends into the valve bonnet.

As throttle construction valves begin to open, the disc shoes slide up inclines at the bottom of each body rail lifting and holding the disc ring face out of contact with the body seat ring face. In closing, the disc shoes slide down the tracks and the inclines at the bottom of each track until the disc ring face is in contact with the body seat ring face thus permitting the wedging of the discs in the closed position.

The three disc shoes register with the three body rails and are in contact with one another while the valve is unseated and in a partially open position. The non-ferrous metal disc shoes and body rails are designed with ample bearing capacity. Throttle and/or beamed waterway construction thus assures long gate valve service life under severe operating conditions.

Construction and Operation Features

The several basic types of gate valves briefly described are usually manually operated, employing key wrenches, handwheels, chain wheels or floorstands. They can, however, be cylinder or electric motor operated. Manually operated gate valves are either of the non-rising stem (Figure 1) or of the rising stem type (Figure 2). The operating threads of non-rising stem valves are always in contact with the fluid or gas in the valve body, whereas the operating threads of rising stem valves are outside of the valve body and, therefore, are not in contact with the fluid.

Gate valve constructions and refinements introduced during the twentieth century include "O" Ring pressure seals, position indicators for non-rising stem valves, valve body clean-outs, gearing, by-pass valve assemblies, tracks, rollers and scrapers, square bottom case and disc, throttle and beamed waterway construction.

"O" ring pressure seals replace conventional valve stem packing (Figure 5) and reduce maintenance to a minimum. Position indicators for non-rising stem valves establish visibly at

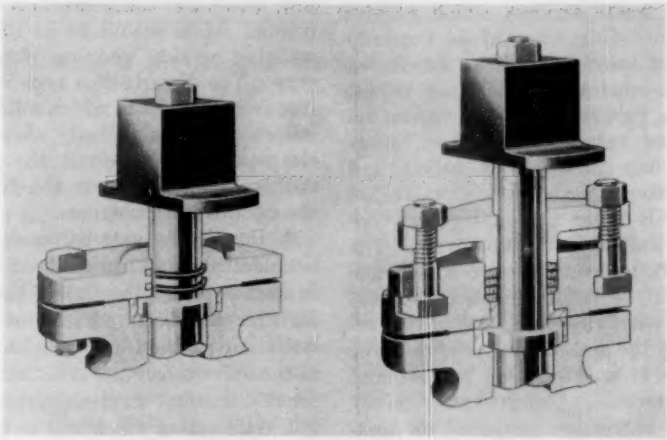

FIGURE 5.—Exposed view of "O" ring seal plate (left) and conventional stuffing box and packing (right) used on non-rising stem valves.

all times the position of the valve discs or wedges. Rising stem valves do not require position indicators as the valve stem serves as an indicator.

Valve body clean-outs should be provided on all gate valves which will be in service in piping conveying suspended solids in solution. The clean-outs should be inspected periodically to flush the suspended solids from the valve body and from contact with the internal operating parts of the valve.

Gearing and by-pass valves are used to reduce the torque required to operate large valves. By-pass valves are also used to fill the line on both sides of the valve thus balancing the pressure before the main valve is opened. Enclosed gearing is normally applied to valves 14 in. and larger in diameter, which in service will be used at or near the working pressure rating.

Vertical valves are normally equipped with spur gearing; horizontal valves with bevel gearing. Horizontal valves installed on edge in horizontal piping should be equipped with bronze tracks, rollers and scrapers. The bronze tracks and rollers carry the weight of the discs during the operating cycle.

Many types of valves are available and employed in connection with the numerous and varied sewage and industrial waste services which require valves including gate valves, butterfly valves, diaphragm valves, lubricated and non-lubricated type plug valves, eccentric rubber-faced plug valves, rotary cone valves, mud valves, shear valves, flap valves, foot valves and sluice gates. Each type of valve has limitations from an employment, operating and economic standpoint. The full operating conditions to which each valve will be subjected should be examined carefully and should be reflected in the determination of the type of valve to use for each control and shut-off service. Unfortunately, many types of valves are subjected on occasion to operating conditions beyond their design and recommended limits.

Operating pressure frequently is a deceiving criterion when it is used to determine the test pressure rating of the valve to be employed. More important criteria include frequency of operation, speed of operation, velocity and the nature of the fluid from the standpoint of the pH and the suspended solids content.

The minimum desired service life of valves is generally considered to be 50 years. To insure long service life with minimum maintenance, maximum valve construction should be considered with respect to pressure rating, shell thickness, trim composition and trim dimension.

Sewage and Industrial Waste Service Applications

1. Non-rising stem gate valves should not be used in piping conveying sludge or heavy concentrations of suspended solids because the operating threads would be in contact with the fluid and solids in the valve body. Rising stem gate valves should be used in exposed accessible piping. The operating threads are outside of the valve and not in contact with the fluid. The valve stem serves as a position indicator.

2. Gate valves used in lines conveying moderate concentrations of suspended solids should be of the center wedging or side wedging, double disc type. The discs in this type valve are free from pockets in which solids would collect. The valve body should have clean-outs which permit the periodic flushing of solids from the body and the operating mechanism.

3. Double disc gate valves should not be installed in an upside-down position in horizontal piping unless the manufacturer has been advised of the installation position and provision made in the valve construction to compensate for this unusual service requirement.

4. Gate valves which will be operated frequently and when in service will be either in a fully open or closed position

should be of the square bottom case and disc type.

5. Gate valves subjected to frequent operation, substantial unbalanced pressure, high velocity and used to control flow should be of the beamed waterway or throttle track type.

6. Gate valves, sizes 14-in. and larger, should be equipped with gearing if the valves will be subject to an operating pressure at or near the recommended working pressure. These large gate valves should be equipped with by-pass valves permitting pressure equalization before the main valve is opened.

7. Valves placed in inaccessible locations should be operated by a floor-stand, chain wheel, cylinder or electric motor. Cylinder operation is economical because the source of power is usually provided from existing installations. Cast iron, bronze-lined hydraulic cylinders should be used as they are extremely rigid and assure long service life under severe operating conditions.

8. The valve manufacturers should be advised concerning the type of service and the characteristics of the fluid in the piping so that the valve will be of appropriate construction and equipped with the proper metal trim.

9. Gate valves should be operated at least twice each year. Valve stuffing boxes should be checked and adjusted at least four times a year. Proper and timely maintenance will insure long, efficient gate valve service life.

INTERESTING EXTRACTS FROM OPERATION REPORTS

"This is a land of plenty, to be sure—but we can't slow down on the job and have that plenty."—R. G. Mowbray

Attention operators! It will be appreciated if copies of all annual reports of both large and small plants are sent to the Federation office for abstracting in this section of the Journal. Carbon copies of typewritten reports will be promptly returned on request. Please direct annual reports to: Federation of Sewage and Industrial Wastes Associations, 4435 Wisconsin Ave., N.W., Washington 16, D. C.

Annual Report of the Jackson, Mich., Sewage Treatment Plant for the Fiscal Year Ending June 30, 1954 *

By R. A. Greene, *Superintendent*

The average flow of 9.09 m.g.d. was the third highest average daily flow in the 17 years of operation and represents an increase of .49 m.g.d. over the previous year. Overhauling of the settled sewage pumps increased the pumping capacity thereby allowing treatment of an increased flow from rain storms, melting ice and snow, and infiltration. There was no indication that the increase in flow was caused by domestic and industrial use of water.

A combined sewer system serves the citizens of Jackson. The previous record high daily flow occurred in 1942–43 when the rainfall for that period was 42.3 in. During the fiscal year 1954 the rainfall was 33.38 in. The dry weather flow also has shown an annual increase.

The total quantity of both suspended solids and B.O.D. removed increased

* For last previous extract see This Journal, 26, 1, 105 (Jan., 1954).

slightly above the previous year. The activated sludge process operated without upset and produced good results. Suspended solids and B.O.D. removal averaged 94 per cent.

Industrial Wastes

There were no operating difficulties caused by industrial wastes. The passing of the sewer ordinance in 1953 has aided considerably in controlling the use of the sewer system. Previously harmful concentrations of wastes from the electroplating industries caused upsets in the aeration units. There were no instances during the year when concentrations of industrial wastes reached the proportions of previous years, although there were some cases when the concentrations exceeded the maximum limitations set forth in the ordinance. Industries as a whole were cooperating very well.

Equipment

The comminutors did a creditable job in shredding coarse materials, although the lower baskets are badly worn by sand and grit and will need replacement in the next two or three years. Grit channels were used at all times and would have functioned better if they had preceded the comminutors.

Sedimentation and aeration tanks operated with normal efficiency. The diffuser plates installed in 1952 have shown no signs of becoming clogged. The practice of forcing two to three times the normal amount of air through them four to six times per year appears to keep the plates clean. Previous to replacing the diffusers it had been necessary to clean the plates every five years. With the practice of blowing the plates it is hoped to lengthen the interval between cleanings.

Sludge Digestion

Operation of the six digesters as two three-stage units was continued. Raw sludge was pumped to the two units on alternate days. The digestion process was completed in the first and second stages with the third stage serving principally for sludge concentration. A reduction in the total volume of gas produced in the digestion process continues as it has during the past few years. Only possible reason is the increased use of detergents by domestic and industrial water users.

Sludge is air dried in sludge beds and shallow lagoons. All sludge produced is removed by state prisoners for use on prison farms as a soil conditioner. Sludge is also available to the general public without charge from a stockpile on the plant grounds.

Plant Maintenance

Preventive maintenance was actively undertaken on a regular schedule. Most repairs resulted from normal wear. New impellers, wearing rings and sleeves were installed in three sewage pumps. A considerable amount of interior and exterior repainting was done. Sludge drying beds were resanded and the shallow lagoons were graded.

The treatment plant site occupies 55 acres between U. S. Highway No. 127 and the Grand River. As a result of its location the plant grounds have been enjoying an increased use from the public.

An additional section of the grounds was developed as a picnic area and 12 additional roadside tables were purchased to accommodate the crowds that gather to use the picnic area. Continued efforts have been made to keep the lawns, shrubs and trees attractive.

The roads inside the plant grounds were all seal coated during the year. A new tractor was purchased and equipped with three 30-in. gang mowers. This piece of equipment has proven to be more economical and much more rugged than the power mowers used previously.

Table I summarizes operation data for the fiscal year 1953–54.

TABLE I.—Summary of Operation Data for the Jackson (Mich.) Sewage Treatment Plant for the Year Ending June 30, 1954

Item	Average
Connected population (est.)	50,000
Rainfall, total (in.)	33.38
Sewage treated:	
M.g.	3,318.92
M.g.d.	9.09
G.p.c.d.	181
Grit (cu. ft./mg.)	1.0
Sewage analyses:	
Suspended solids:	
Raw (p.p.m.)	207
Settled (p.p.m.)	169
Final (p.p.m.)	14
Reduction (%)	93
B.O.D.:	
Raw (p.p.m.)	149
Settled (p.p.m.)	112
Final (p.p.m.)	7
Reduction (%)	95
Lb. per cap. per day in raw	0.22
Dissolved oxygen (p.p.m.):	
Raw	2.5
Settled	3.1
Final	5.4
Oxygen consumed (p.p.m.):	
Raw	49
Settled	41.2
Final	11.4
Organic nitrogen (p.p.m.):	
Raw	8.4
Settled	8.2
Final	3.9
Ammonia nitrogen (p.p.m.):	
Raw	11.4
Settled	12.0
Final	12.0
Nitrite-nitrate nitrogen (p.p.m.):	
Final	0.0
Aerators:	
Mixed liquor susp. sol. (p.p.m.)	2,100
Sludge index	38
Air used (cu. ft. per gal.)	0.7
Kilowatt hours per m.g.	491
Return sludge (%)	20
Primary sludge:	
Solids (%)	4.9
Volatile solids (%)	58
Solids removed, dry lb./m.g.	1,610.4
Volume (gal./m.g.)	3,988
Digested sludge:	
Solids (%)	9.2
Volatile solids (%)	44
pH	7.0
Dry sludge removed:	
(cu. yd.)	7,590
Gas production:	
Total (1,000 cu. ft.)	17,392
Gas per lb. vol. sol. digested (cu. ft.)	13.7
Cu. ft. per capita per day	0.95
Cost data ($):	
Cost of operation	88,663.67
Revenue bonds (retirement and interest)	40,147.75
Collection (billings, etc.)	19,361.10
New equipment	3,311.57
Lift stations and sewer maintenance	32,894.40
Insurance	465.05
Taxes	12,000.00
Total	196,843.49
Cost per m.g. (fixed charges included)	45.78
Cost per m.g. (less fixed charges)	32.55

First Annual Report of the Port Huron, Mich., Sewage Treatment Plant for the Fiscal Year Ended June 30, 1954

By L. D. Working, *Superintendent*

The first sewage was admitted to the 15 m.g.d. primary treatment plant on June 16, 1952 with the final cut-in made the end of December. Inasmuch as this is the first fiscal year of full-scale operation the report covers activities from the initial flow of sewage, while analytical results are for the period July 1, 1953 to June 30, 1954.

Sewers and Lift Stations

Seven sewage pump stations, two inverted syphon river crossings and 6.2 miles of interceptor sewers collect the

sewage along both banks of the Black and St. Clair rivers. Diversion chambers fitted with orifice plates direct most of the storm water to the rivers, since the original lateral sewers were constructed as combined sewers. Three of the lift stations are equipped with mechanically cleaned bar screens. Five of the lift stations are new and were placed in operation the end of October, 1952, with the last station completed by the end of the year.

A delay of three months in getting the new pump stations in service was caused by bearing failures in the pumps. All 21 pumps were dismantled and checked, and most of the bearings in the pumps were rusted and corroded due to moisture and long idleness. Thrust bearings in the pumps in two of the pump stations were found to be damaged by improper erection procedure.

Treatment Plant

The raw sewage passes through mechanically cleaned bar screens where the reducible screenings are ground and returned to the sewage. From the wet well the screened sewage is lifted to two mechanically cleaned grit chambers.

Eight rectangular settling tanks provide for solids removal. Settled solids are pumped to four fixed-cover, 55-ft. diameter heated digesters. The digested sludge can be dried on enclosed drying beds or on a rotary drum vacuum filter. Digester gas is utilized for heating purposes and to operate two 4-m.g.d. gas engine raw sewage pumps.

The chlorinated settled effluent discharges to the St. Clair River at a depth of 30 ft. and 140 ft. off shore.

Garbage Disposal

Incorporated in the design are facilities for the future processing of the garbage from the entire city. Raw garbage will be emptied on a sorting deck and ungrindable materials (cans, glass, crockery) will be removed. The ground garbage will be mixed with water in pits beneath the grinders and air agitation will settle the eggshells, glass and grit to the pit bottom. The garbage then will be pumped to the digesters and the grit to the grit chambers.

Digester Operation

Two of the four digesters were filled with raw sewage on June 16, 1952 and heating was started immediately. By June 24, digester temperature was 80° F. and small quantities of raw sludge were added daily. The digestion process showed no sign of foaming and by July 11 both digesters were operating at 90° F. and producing a small quantity of burnable gas.

Personnel

The treatment plant and lift stations are operated on a 24-hr. schedule by a superintendent, chief operator, four shift operators, a pumping station operator, two laborers and a janitor. All maintenance, except work requiring specialized or heavy equipment, is performed by plant personnel. Preventative maintenance is of predominant importance.

During initial reduced-scale operation, equipment was observed closely and necessary adjustments made. Because of the plant size and operating schedule each operator was individually trained in all phases of operation and maintenance, including laboratory procedures. Many months of experience are required before an operator acquires proficiency in all phases of his work; consequently, the positions were carefully filled from those who passed a civil service examination.

Initial Problems

Extremely high river levels during 1952 and 1953 permitted river water to overflow some of the diversion chamber dams. Planks were installed to

raise the elevation of the dams. Some flooding of low basements occurred during rainstorms.

Considerable difficulty was experienced with almost all of the equipment, due primarily to careless and sloppy assembly and fabrication. There were a few instances of failure of equipment due to defective material or poor design. In some cases it was necessary to completely reset equipment in order to align it properly. Where major changes, alterations or repairs were necessary the equipment manufacturer furnished erection engineers for supervision and assistance. Attempts by the equipment manufacturer to charge the city for these services and parts were unsuccessful.

Much time, however, was spent by plant personnel in assisting with the work of making changes and alterations.

A total of 34 modifications or additions were made during the final stages of construction. These additions will materially improve the operating efficiency and safety of the treatment plant.

The total cost of the sewage treatment plant, lift stations and interceptor sewers was approximately $3,000,000. Financing is by means of general obligation and revenue bonds. Receipts are collected from sewer service charges.

Operating data for the fiscal year 1953–54 is summarized in Table III.

TABLE III.—Summary of Operation Data for the Port Huron (Mich.), Sewage Treatment Plant for the Fiscal Year 1953–54

Item	Average
Connected population (est.)	36,000
Sewage flow (m.g.):	
Total	3,275.1
Daily	8.8
Grit:	
Cu. ft./day	31.8
Cu. ft./m.g.	3.6
Suspended solids:	
Influent (p.p.m.)	144
Effluent (p.p.m.)	64
Reduction (%)	56
Settleable solids:	
Influent (ml./l.)	1.3
Effluent (ml./l.)	.1
Reduction (%)	93
Primary sludge:	
Volume (gal./day)	16,101
Total solids, dry (lb./day)	5,446
Total solids (%)	3.7
Volatile solids (%)	55
pH	6.6
Digested sludge:	
Total volume (1,000 gal.)	351
Total solids, dry (lb./day)	1,004
Total solids (%)	10.5
Volatile solids (%)	38
pH	7.0
Gas production:	
Volume (cu. ft./day)	13,180
Cu. ft./lb. vol. solids	4.4
Cu. ft./capita	0.38
Chlorine:	
Used (lb./day)	314
Residual (p.p.m.)	0.68
Operating expense ($):	
Payroll	43,902.52
Utilities	8,424.35
Repairs	2,503.11
Supplies	2,348.18
Adm. and gen.	1,823.62
Chemicals	4,577.70
Insurance	2,125.12
Pension fund	1,756.10
Total	67,460.70

INGENIOUS ANSWER TO TRENCH CAVE-INS *

The question of what to do about trench cave-ins is old and troublesome. Too many men are injured or killed each year by the collapsing walls of deep trenches. More and more states have enacted laws that make timbering mandatory. And, even aside from the human problem, cave-ins are a constant threat to the progress of a job involving deep trenches in unstable soil.

Yet timbering is expensive and time consuming. Contractors and sanitary engineers have struggled with the problem for years.

A Simple Answer

The answer that has been put into effect by the Engineering Department of the city of Marshfield, Wis. in simple—it has the ring of a real answer.

The device is a stoutly constructed portable steel crib (Figure 1), open at the bottom with ample room inside for pipe layers to work. The cost is low.

Easily Portable

The Marshfield "box" consists of a welded and heavily braced skeleton of 3-in. steel pipe, covered on the two sides with ¼-in. steel plate. It has an inside working width of 30 in., is 12 ft. long and 9 ft. high, with attachable side plates that extend the height to 12 ft. These dimensions are, of course, designed to the Marshfield working requirements and could easily be varied.

The box is specially designed to work in conjunction with a back-hoe. A heavy chain easily converts the dipper stick to derrick service for lowering the box into the ditch, moving forward as work progresses, and removing it when the ditch is finished.

When the pipe layers are laying pipe and making joints, inside the box, the back-hoe digs another box length ahead. Usually, there is little or no waiting involved.

FIGURE 1.—Portable steel trench crib.

Safer than Timbering

Not only does this trench crib reduce timbering time and cost to insignificance, but it is a permanent assurance against improper timbering, sometimes a danger in itself. The top can be covered with heavy planks, rendering the box quite safe even in trenches extending several feet above the crib top.

* Reproducted through the courtesy of the Clay Products Assn., Chicago, Ill.

NOTES ON THE DETERMINATION OF SUSPENDED SOLIDS

By Jerome Degen and Frederick E. Nussberger

Assistant Chemist and Senior Chemist, respectively, Department of Public Works, New York City, New York

Standard Methods for the Examination of Water, Sewage, and Industrial Wastes, Tenth Edition (1955), gives the following procedure for determining suspended solids in sewage. "Measure out 50 to 100 ml. well mixed sample with a wide tip pipet or measuring flask and filter through the weighed Gooch crucible using suction. Wash with distilled water, dry at 103° C. for one hour, cool in desiccator and weigh." It was suspected that occasionally high suspended solids values were due to inadequate washing with distilled water. Accordingly, to investigate the proper procedure, a series of experiments was made.

Effect of Dissolved Solids

A solution containing 6.59 g. of sodium chloride per liter, equivalent to 4,000 p.p.m. of chloride, was prepared. Five 100 ml. portions of this were filtered through five ignited Gooch crucibles. After filtering the crucibles were dried without washing, cooled in a desiccator, and weighed. The average results indicated that 0.0013 g. of salt was held in the mat. This is equivalent to 0.197 ml. of the original salt solution. The average weight of the mats in the crucibles was 0.1708 g. Accordingly, 1.15 ml. of the salt solution, or 0.0076 g. of dry salt, was retained per gram of asbestos mat.

To investigate the best procedure for washing the Gooch crucibles 10 crucibles were prepared and divided into two sets of five crucibles each. Then 100 ml. portions of the salt solution were poured through each of the first five crucibles followed by one 20 ml. portion of distilled water. The second set of five crucibles was treated as the first except that each crucible was washed with two 10 ml. portions of distilled water. The average gains in weight were 0.3 mg. for the first set and 0.0 mg. for the second set. The results of this experiment and of other experiments, in which lesser and greater amounts of asbestos were used in preparing the crucible mats, indicated that two successive portions of 10 ml. water each were sufficient to wash the filter mat substantially free of salt.

To demonstrate the effect of washing, using a sample of sewage, the following experiment was made. A sample of raw sewage containing an appreciable amount of sea water was allowed to settle and the supernatant decanted. The supernatant was analysed and found to contain 1,950 p.p.m. chlorides and 4,070 p.p.m. total solids. A series of 10 ignited Gooch crucibles was prepared and divided into two sets. One hundred milliliter portions of the supernatant were poured through each crucible in the first set of five crucibles and these crucibles were then placed in the drying oven without washing. Then 100 ml. portions of the supernatant were filtered through the second set of five crucibles, followed by two 10 ml. portions of distilled water. These crucibles were then dried in the oven, placed in a dessicator and weighed. They were then ignited at 600° C., cooled, placed in a desiccator and weighed again. The results for suspended solids and per cent volatile suspended solids are as follows:

Without washing:	91 p.p.m. suspended solids
	79.0 per cent volatile

With washing: 72 p.p.m. suspended solids
82.3 per cent volatile

To investigate whether or not the error caused by the dissolved salts could be minimized, another series of determinations was made. The crucibles were allowed to remain under suction longer, and an additional step of blotting the bottoms of the crucibles with filter paper before placing in the drying oven was undertaken. It was found that, without washing, the errors introduced by the dissolved salts were thereby reduced about 20 per cent.

Effect of Asbestos

Another source of error in the suspended solids determination may arise in the preparation of the Gooch crucible and its mat. The characteristics of the particular shredded asbestos used for preparing the mat and the size of the perforations in the crucible are particularly important. It was suspected that errors might be due to a loss of asbestos from the mat of the Gooch crucible during the steps of filtering the sample through the crucible and the subsequent washing of the mat plus suspended solids with distilled water. For this reason an experiment was set up to determine: (a) if there was any loss of asbestos from the mat during the filtration and subsequent washing; and (b) to what extent the loss, if any, might affect the results of the suspended solids and volatile suspended solids determinations.

In this experiment five sets of five crucibles each were prepared, using three different brands of asbestos and two different boxes of a fourth brand. The standard method does not specify the size of the perforations of the Gooch crucibles. Coors No. 3, having holes 0.7 mm. in diameter, are generally used. In this experiment Coors No. 3B crucibles, having holes of 1.0 mm. diameter, were used. The mats were prepared by producing a slurry of asbestos, using 900 ml. of distilled water to about 7 g. of the asbestos to be tested. The crucibles were filled nearly to the top with the slurry.

After the water from the slurry had percolated through the crucibles it was assumed that the heavier and coarser asbestos particles had settled more rapidly and that the finer particles were deposited largely on top of the coarser particles. The crucibles were then transferred to the suction filtration setup, suction was applied at 15 to 20 in. of mercury, and the remainder of the water was removed. Each crucible was then washed with distilled water and drained under suction three times, allowing complete drainage between each addition of wash water. The crucibles were then placed in a drying oven and dried at 103° C., removed from the oven and allowed to cool somewhat before being placed in a desiccator for one hour. After weighing, they were ignited at 600° C. for one hour, cooled, placed in a desiccator for one hour, and weighed. A loss in weight was taken as the loss on ignition.

The next step was to filter 100 ml. of distilled water through each of the crucibles, using a suction of 15 to 20 in. of mercury. Then two 10 ml. portions of distilled water were filtered through. Each portion of water was allowed to drain through completely before filtering through the next portion of water. The crucibles were then dried in an oven at 103° C., then ignited at 600° C. for one hour, cooled, placed in a desiccator for one hour, and weighed.

The difference between this final weight and the initial weight was considered to be the loss of asbestos mat such as might occur during an actual determination of suspended solids and volatile suspended solids. A summary of the averaged results obtained is given in Table I.

The loss of non-volatile asbestos from

TABLE I.—Loss of Asbestos During Filtration and Washing

Item	Brand of Asbestos				
	A	B	C	D1	D2
Weight of asbestos mat (g.)	0.0930	0.1490	0.1509	0.1182	0.1112
Loss of weight of mat after ignition (g.)	0.0002	0.0004	0.0006	0.0005	0.0002
Loss of weight of mat after filtration of distilled water	0.0012	0.0008	0.0007	0.0007	0.0044

a mat during the filtration step produces a falsely high percentage volatile result. If, for example, a poor grade of asbestos mat were used which lost 0.0010 g. during the process of filtering and washing a 100 ml. portion of sample, it will be seen that a sample of raw sewage containing 150 p.p.m. suspended solids and 80 per cent volatile suspended solids, may be found to have only 140 p.p.m. suspended solids and 85.7 per cent volatile suspended solids. Similarly, a sample of primary effluent containing 50 p.p.m. suspended solids and 80 per cent volatile suspended solids, would be found to have 40 p.p.m. suspended solids and 100 per cent volatile suspended solids. In the same way, a final effluent sample containing 15 p.p.m. suspended solids may, under the same conditions, be found to have only 5 p.p.m. suspended solids.

Effect of Crucible Perforation Size

In order to determine whether the size of the perforations of the Gooch crucibles effected loss of weight from the asbestos mats during the filtration and subsequent washing operations, two sets of five crucibles each were prepared in an identical manner. The first set of crucibles were Coors No. 3B having perforations 1.0 mm. in diameter. The second set were Coors No. 3 having perforations 0.7 mm. in diameter. Asbestos brand "B" acid washed medium fiber was used in all 10 crucibles.

TABLE II.—Relation of Weight Loss of Asbestos to Crucible Perforation Size

Coors No.	Perforation Size (mm.)	Loss of weight after filtration of distilled water (g.)
3B	1.0	0.0008
3	0.7	0.0004

A summary of the averaged results is given in Table II.

In these tests a vacuum of 15 to 20 in. of mercury was used both for preparing the Gooch crucibles and for filtering the samples. Experience indicated that, to minimize the errors due to loss of asbestos mat, the vacuum used for filtering a sample should be less than the vacuum used in preparing the asbestos mat.

Other experiments indicated that it is possible to reclaim the asbestos from used Gooch filter mats by treating with hydrochloric acid and washing with distilled water. The fines are partly removed in the process and quite satisfactory mats can be prepared from the remaining fibers.

Conclusions

In general, errors in the determination of suspended solids and volatile suspended solids by the Gooch crucible method will be minimized if the following procedures are followed:

1. A Gooch crucible having small perforations should be used.
2. The salts dissolved in the sewage sample should be washed out of the mat following filtration of the sample.

3. The asbestos used in preparing the mat should be a good grade as determined by a trial test.

4. Filtration of the sample should be made at a lower vacuum than that used in preparing the asbestos mat.

5. Preparation of the asbestos mat to avoid loss of asbestos during the filtration and washing of a sample.

Acknowledgment

The authors wish to thank Mr. Robert Shapiro, Chief of Laboratories, for direction.

TIPS AND QUIPS

Atomic Sterilization

Research in the sterilization of sewage utilizing nuclear radiation has recently been announced by the city of Los Angeles. The work will be done by a commercial group interested in the industrial application of radioactivity.

Studies will be undertaken to evaluate the effectiveness of radioactivity in conjunction with conventional methods of sewage treatment. Some investigators have considered the feasibility of disposing of raw sewage merely by sterilizing it. Such a possibility has interesting ramifications.

Pilot Pressure Filter

A fully enclosed minature pressure filter is available for filtration research and for the determination of filtration characteristics. Specific filtering rates in gallons per minute per square foot can be obtained since the filter may be used with one, two or three vertical filter leaves. The net area of the three leaves is 1 sq. ft. The unit is designed for precoat filtration using diatomaceous earth or other filtering aids.

The filter shell measures 6.5 in. in diameter and has an over-all height of 16 in. This size makes it readily portable and ideal for laboratory or pilot-plant use. Information on the unit may be obtained from American Plant Equipment Co., Manufacturing Division, 701 Spring St., Elizabeth 4, N. J., by requesting information on the "Pronto-Junior."

"Hot" Bricks

Safe disposal of radioactive wastes can be achieved by sealing the radioactive residues in glazed bricks. Research, using the clay mineral known as montmorillonite, was conducted at the USAF Institute of Technology, Wright-Patterson Air Force Base, Ohio.

The disposal process is visualized as mixing the radioactive materials with the clay slurry or allowing the radioactive solution to trickle through a column of clay. The radioactive particles would become enmeshed in the clay. Excess water would be removed by draining, filtering or centrifuging, and the clay glazed into bricks. Glazing would produce a package which could be impervious to leaching and thus permit safe disposal in the ocean or underground.

All Weather Pipe Wrap Tape

A pressure-sensitive, polyvinyl chloride pipe wrap tape featuring increased adhesion, and employing a built-in corrosion inhibitor has been introduced by Minnesota Mining and Manufacturing Co., 900 Fauquier St., Saint Paul 6, Minn.

As handy as "Scotch tape," this improved "Scotchrap" should be useful for wrapping above-ground and buried pipes wherever corrosion is a problem. It is easy to use, particularly for resealing or repair jobs. The tape comes in 100-ft. rolls and a variety of widths.

Cold Weather Accidents *

Falls, burns and asphyxiation are the principle causes of accidents during the winter months, excluding motor vehicle accidents.

Fatal falls are at a maximum, but their seasonal rise is less than for any of the other leading causes of accidental death. In 1951–53, February was the peak for falls, but the death toll in that month was only 8 per cent above the annual average.

As might be expected, deaths resulting from fires and burns by other means occur with greatest frequency during the cold weather, and are associated with the increased use of heating facilities. The loss of life during the winter is approximately 1½ times the annual average.

The rise in mortality from poisonous gas with the onset of colder weather is likewise associated with the increased use of heating facilities. In addition, an appreciable number of deaths are due to the running of an automobile motor in a closed garage or in cars parked on the street or highway. Gas poisoning shows a wide seasonal variation, the mortality in December and January being nearly eight times that in July.

Check your treatment plant facilities today for those hazards causing crippling or fatal falls, burns or asphyxiation.

Emergency First Aid Chart

A completely revised edition of the Fisher Laboratory Emergency Chart brings an up-to-date comprehensive, condensed, alphabetical compilation of emergency treatments for specific poisons and particular accidents.

Emergency aid for burns and scalds, cuts, collapse, toxic headaches, electric shock and poisoning are briefly described. General rules, valuable at any time and place, are clearly outlined. Illustrated is the Red Cross recommended back pressure-arm lift method of artificial respiration.

The chart, 18 in. by 27 in., is available without charge to any established laboratory from Fisher Scientific Co., 717 Forbes St., Pittsburgh 19, Pa.

Some Days It Rains †

Everything was too quiet at the Boise, Idaho, sewage treatment plant. No griping, just a continuous purr from the operating equipment, and then things began to pop.

1. A leak in the 30-in. concrete pipe from the raw sewage pump to the grit basin required extensive excavating.

2. A 4 ft. length of 4-in. timber wedged into the comminutor, breaking the drum, trust bearing, teeth and other parts to the extent of $1,790 for repairs.

3. The 10 h.p. return sludge pump began to growl, requiring replacement of the bearing.

4. Finally, the rubber-lined empeller of the raw sewage pump broke as the empeller was being removed from the shaft for relining.

Fortunately, no one was hurt, only one day of by-passing was necessary and effluent quality remained excellent. The Boise Council was "budget-wise" and had set aside sufficient funds to cover emergencies. This was commendable economic planning for the health and welfare of the community. Every sewage treatment plant budget should have a "rainy day" fund.

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* Condensed from *Statistical Bulletin*, Metropolitan Life Insurance Co. (Nov., 1955).

† Extracted from *Sewage Gas*, sewage treatment plant newsletter, Boise, Idaho.

Reviews and Abstracts*

Solving Pollution Problems Through Cooperation. By J. M. Jarrett. *Proc. Amer. Soc. Civil Eng.*, 81, Sep. No. 642 (Mar., 1955).

In planning the solution of stream pollution problems more attention should be given to the social and economic factors involved. The engineer has often failed in the interpretation of technical facts for the public. The author discusses the needs for better cooperation and then shows how the North Carolina State Board of Health has acted to bring about cooperation with other agencies and to educate industrial leaders and the public. W. A. Hasfurther

Capitalizing on Municipal Wastes by Composting. By Richard P. Storroff. *Proc. Amer. Soc. Civil Eng.*, 80, Sep. No. 545 (Nov., 1954).

This paper covers the operational aspects of composting garbage for a municipality. At Oakland where open-air windrowed piles are used, there is a minimum of equipment and the new plant designed to handle 265 tons of refuse a day is returning a good profit. Plant costs and revenues are tabulated. W. A. Hasfurther

Toxicity of Amines for Bacteria. By R. A. MacLeod and Eva Onofrey. Pacific Biological Station and Pacific Fisheries Experimental Station, Fisheries Research Board of Canada. Progress Report 98, p. 22 (April, 1954).

It was found that high concentrations of potassium ions were able to overcome the toxicity of triethanolamine to bacteria. Other ions such as Na, NH_4, Mg and Ca had a similar effect. A level of amine which was toxic in a neutral medium was non-toxic under slightly acid conditions. Other amines such as triethylamine and tri-n-propylamine were also effected by inorganic ions in the same way.

It is believed that the action entails a combination of the positive amine ion with a negative site on an enzyme or on the cell of the bacteria. Positively charged ions combining with this site would protect the organism against the action of amines.

H. Heukelekian

Composting and Grinding: Report of Subcommittee of the Committee on Refuse Collection and Disposal. *Proc. Amer. Soc. Civil. Eng.*, 80, Sep. No. 556 (Nov., 1954).

Aerobic composting processes are being tried in this country but operations are still in the experimental stage. The present status of the Frazer-Eweson Process, the Compost Corp. of America, the Atloona FAM, Inc., and Growth, Inc., operations are mentioned. Full-scale works of the first three are being revised to reduce costs.

Some statistics on the use of home garbage grinders are given along with the basic design data and operation experience at the Jasper, Ind. sewage treatment plant where 75 per cent of the homes have grinders. Experience there and elsewhere shows that garbage solids and B.O.D. are readily removed in normal treatment processes and that increased digestion capacity (5 cu. ft. per capita for primary solids and garbage) is necessary. W. A. Hasfurther

Sanitary Engineering Aspects of Nuclear Energy: Committee Progress Report. *Proc. Amer. Soc. Civil Eng.*, 81, Sep. No. 646 (Mar., 1955).

The committee warns the engineer and waterworks operator of technical knowledge he must learn before a method for treating radioactive wastes can be provided. Specific isotopes must be considered and most of the paper is a review of 43 references on treatment, handling and measurement of radioactive materials.

W. A. Hasfurther

* Please send to Federation headquarters all periodicals, bulletins, special reports, etc., which might be suitable for abstracting in This Journal. Publications of public health departments, stream pollution control agencies, research organizations, and educational institutions are particularly desired. Address such material: Federation of Sewage and Industrial Wastes Assns., 4435 Wisconsin Ave., N.W., Washington 16, D. C.

Rivers Department, City Manchester, England. Report for the Two Years Ended March 31, 1954. By J. McNicholas.

No major change occurred in the operation of the Davyhulme Works during 1953–54. The B.O.D. load was 151,600 lb. per day. The increasing use of synthetic detergents has caused excessive foaming.

Much of the treatment plant is obsolete and the primary and secondary contact beds will be scrapped shortly as they effect little purification.

The activated sludge plants have not produced consistently satisfactory effluents during the past two years, although the bio-aeration (Sheffield) unit has treated an increased volume with better quality effluent than either the Simplex or diffused air plants. The average power requirement has been 26.4 h.p. per m.g. (Imp.) The conversion of diffused air units from plate to dome-type has continued.

Sludge is disposed at sea, at a cost of 4 shillings per wet ton.

The work of the Department includes: surveillance of rivers and streams; surveillance of trade effluents from 500 establishments; operation of sewage works at Davyhulme and Withington; construction and maintenance of 43.76 mi. of sewers; and experimental work. A tax is collected to cover these activities.

TABLE I.—Aerator Effluent Data

Year	Bio-Aeration	Simplex	Diffused Air
B.O.D. (p.p.m.)			
1953	21.9	17.2	24.6
1954	25.5	23.1	31.4
Susp. Solids (p.p.m.)			
1953	24.0	19.0	25.0
1954	27.0	23.0	27.7

When producing effluents of similar character, the various units required the following horsepower and retention periods:

Unit	Hp./m.g.d. (Imp.)	Retention (hr.)
Diffused air	29.9	10
Simplex	27.4	13
Bio-aeration	42.8	17.3

The composition of the combined effluents from all sections of the plant for two years ended March 31, 1954:

	1952–1953	1953–1954
B.O.D., p.p.m.	89.2	83.3
Susp. solids, p.p.m.	41.0	42.0

Experimental work has been conducted on the effect of synthetic detergents on the solution of oxygen in water. A diminution was found in the per cent oxygen dissolved as the amount of detergent added rose from 10 p.p.m. to 1,000 p.p.m.

Experiments were also conducted on the treatment of Withington sewage with aluminoferric prior to aeration.

The report concludes with a detailed tabulation of costs.

Langdon Pearse

BOOKS, REPORTS, PAMPHLETS

The Chemical Industry Facts Book. Second Edition. Manufacturing Chemists Assn., 1625 Eye St., N.W., Washington 6, D. C. 160 pp. Price, $1.00 (1955).

Reference source on the chemical industry and its work. Profusely illustrated.

The Commission on Intergovernmental Relations. A Report to the President for Transmittal to the Congress. U. S. Govt. Printing Office, Washington 25, D. C. 311 pp. Price, $1.25 (June 1955).

The Annual Public Works Report. Special Report No. 19, Amer. Public Works Assn., 1313 East 60th St., Chicago 37, Ill. 15 pp. Price, $1.00 (1955).

A check list useful for the preparation of an annual public works report.

Proceedings 24th Annual Arkansas Water and Sewage Conference and Short Course. Dept. of Civil and Chemical Engineering, Univ. of Arkansas, Fayetteville, Ark. (1955).

Progress Report No. 38, Washington Pollution Control Commission, December 1, 1954–May 31, 1955. Washington Pollution Control Commission, Olympia, Wash. 28 pp. (1955).

News and Notes

Of Persons and Activities in the Field of Hydrowastes

Mark D. Hollis, Assistant Surgeon General and Chief Engineer of the United States Public Health Service was awarded an Honorary Doctor of Science Degree by the University of Florida on January 28, 1956. In his commencement address, Dr. Hollis stressed the deterioration of community water resources resulting from heavy discharges of wastes by growing metropolitan concentration.

Ralph Stone and Company, Consulting Engineers, announce their move to new and larger quarters at 147 San Vicente Blvd., Beverly Hills, Calif. The organization provides complete engineering services for all types of municipal and industrial work.

Harry K. Gidley, associated for 25 years with the Sanitary Engineering Division, West Virginia State Department of Health, has resigned to enter the consulting engineering field in Charleston, W. Va. Mr. Gidley serves the West Virginia Sewage and Industrial Wastes Assn. as Federation Director.

Roy F. Weston, formerly Sanitary Engineer, Atlantic Refining Company, Philadelphia, joined **Professor Wesley Eckenfelder, Jr.** of Manhattan College in a consulting engineering partnership with offices in Newtown Square, Pa., and Leonia, N. J. Mr. Weston is Federation Director for the Pennsylvania Sewage and Industrial Wastes Assn.

Dr. John R. Snell, formerly Professor of Sanitary Engineering, Michigan State College, Lansing, has established a consulting engineering practice under the name of John R. Snell and Associates, East Lansing, Mich.

Another new consultant in the field is **Herbert E. Hudson, Jr.**, formerly Chief Engineer of the Illinois State Water Survey who has joined the firm of Hazen and Sawyer, Engineers, New York, and will be in charge of the Detroit office of that firm. While his work has been mostly in the water world, he is likely to have the problems of abused water in his new undertaking.

The reputation of **Philip F. Morgan**, Professor of Sanitary Engineering at Iowa State University got into the field of politics recently when his friends circulated cards in support of his election as Councilman for the city of Ames, Iowa. The cards which his local supporters had printed read "Fearless Catalytic a vote for Phil is a vote for higher wages . . . lower taxes . . . full parity . . . The Iowa Turnpike and everything else." Professor Morgan's many friends in the sewage and industrial wastes field will continue to watch him for future developments.

Dr. Daniel A. Okun, Professor of Sanitary Engineering, School of Public Health, University of North Carolina at Chapel Hill, has been appointed to the position of Department Head following the retirement of **Dr. Herman Glenn Baity.** Further changes in the department have included the appointment of **Dr. Charles M. Weiss** and **Gilbert L. Kelso** as associate professors of sanitary science.

Dr. Gerald H. Teletzke has joined the faculty of Purdue University as Assistant Professor of Sanitary Engineering, effective with the current semester. Dr. Teletzke was formerly an Instructor of Civil Engineering at the University of Wisconsin. In his new work at Purdue, he will participate in the sanitary engineering and hydraulic teaching at both graduate and undergraduate levels.

Dr. Francis S. Friel, President of Albright & Friel, Inc., Philadelphia, has been cited as Metropolitan Philadelphia's outstanding engineer by the Pennsylvania Society of Professional Engineers. Dr. Friel, who has been an active member of the profession for 40 years has served the field in numerous capacities including the office of President of the Federation in 1946–47.

Edward J. Cleary, Executive Director of the Ohio River Valley Water Sanitation Commission, has announced the promotion of **Bob Horton** to the position of Assistant Director of ORSANCO. Mr. Horton's promotion is in recognition of the responsibilities which he has carried so ably for this organization.

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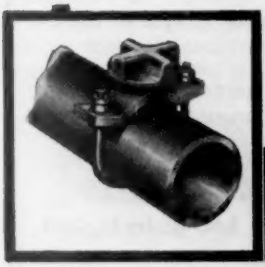

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PACIFIC NORTHWEST SEWAGE AND INDUSTRIAL WASTES ASSOCIATION

The 22nd Annual Meeting of the Pacific Northwest Sewage and Industrial Wastes Association was held in the Empress Hotel, Victoria, B. C., on October 27–29, 1955.

Technical papers presented at the meeting included the following:

"Sewage Disposal and Wastes Treatment in Canada," by J. R. Menzies, Chief, Division of Public Health Engineering, Dept. of National Health and Welfare, Ottawa, Canada.

"Effect of Ground Garbage on the Sewerage System," by R. L. Stockham, District Engineer, Washington State Dept. of Health, Seattle, Wash.

"Sewage Pumping," by M. S. Campbell, Sales Engineer, H. D. Fowler Co., Inc., Seattle, Wash.

"Sludge Pumps and Controls," by Harrison W. Kramer, Consulting Engineer, Carey and Kramer, Seattle, Wash.

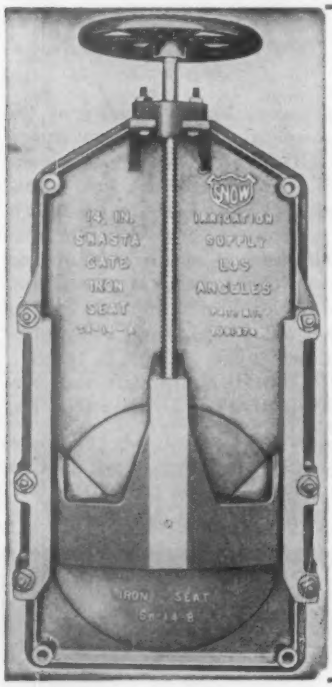
"Oil Refinery Waste Treatment," by C. G. Cortelyou, Manager, Ferndale Refinery, General Petroleum Corp.

"Synthetic Detergents and the Plant Operator," by Richard H. Bogan, Assistant Professor of Civil Engineering, University of Washington.

"Accelerated Sludge Digestion," by Tom H. Forrest, Sanitary Engineer, Chicago Pump Co., Chicago, Ill.

"Preaeration Results," by Richard C. Boughner, Engineer, Washington State Pollution Control Commission.

(*Continued on page 68a*)



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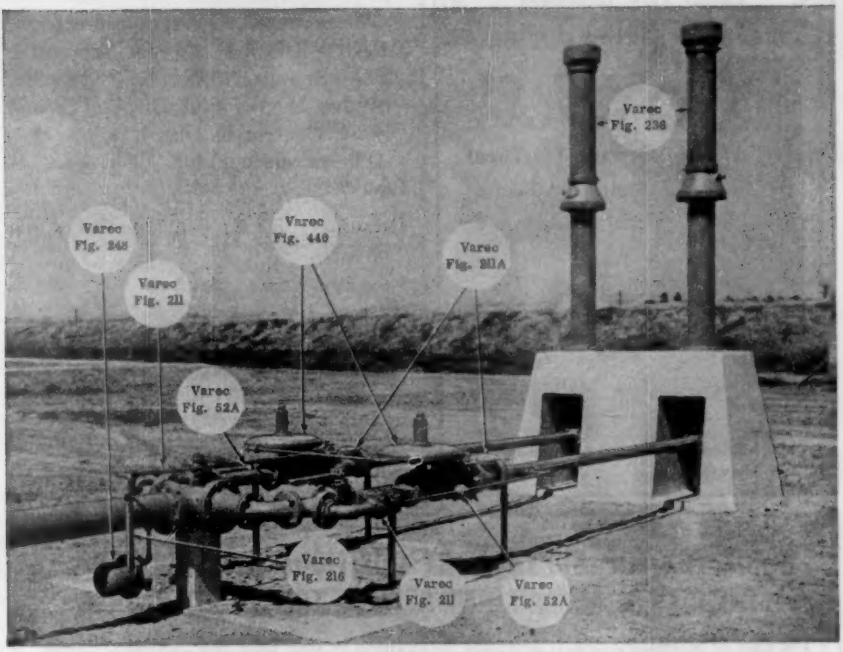

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"Disposal of Radioactive Wastes," by Royal E. Rostenbach, General Electric Co. Radiological Engineering Section, Radiological Sciences Dept., Hanford Atomic Products Operation, Richland, Wash.

There were also symposiums on: "Refuse Disposal by Incineration"; "The Sewage Collection System"; "Sludge Drying and Disposal"; and "Scum Control in Digesters."

Officers elected for 1956 were as follows:

President: E. F. Eldridge, Olympia, Wash.

Vice-President: Andrew J. Wahl, Boise, Ida.

Secretary-Treasurer: Walter W. Saxton, Olympia, Wash.

WALTER W. SAXTON,
Secretary-Treasurer

(*Continued on page 69a*)

SIX (6) TOP SUPERVISORY POSITIONS AVAILABLE IN THE CITY OF MIAMI'S NEW SEWAGE TREATMENT PLANT

The City of Miami, Florida, is accepting applications to staff their new Activated Sludge Sewage Treatment Plant, to be completed during the early part of 1956, as follows:

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ASSISTANT SEWAGE TREATMENT PLANT SUPERINTENDENT. (1) Salary range $6,372–$7,872. Desired Qualifications: 6–8 years previous experience in the operation of complex sewage treatment plants. Education: Degree in Sanitary Engineering or in Civil or Chemical Engineering with courses in sanitary engineering.

SEWAGE TREATMENT PLANT SUPERVISORS. (4) Salary range $5,388–$6,648. Desired Qualifications: At least five (5) years previous experience in the operation of complex sewage treatment plants. Education: Graduation from high school or a vocational school with courses in the chemistry and biology of sewage treatment or related subjects.

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Send complete summary of your education, previous experience and personal data to C. F. Wertz, Director, City of Miami, Department of Water and Sewers, 3342 Pan American Drive, Miami 33, Florida. (All replies treated strictly confidential.)

NEW ENGLAND SEWAGE AND INDUSTRIAL WASTES ASSOCIATION

The Fall Meeting of the New England Sewage and Industrial Wastes Association was held November 3, 1955, at the Woodland Country Club, Auburndale, Mass. The meeting was well attended in spite of the weather; more than 170 were present.

Papers presented at the meeting were as follows:

"Experiences with Synthetic Filter Cloths in Vacuum Filtration of Sludge," by LeRoy W. Van Kleeck, Principal Sanitary Engineer, Connecticut Dept. of Health, Hartford, Conn.

"Sewage Sludge Thickening at the Middleborough, Massachusetts, Biofiltration Plant," by Sterling G. Brisbin, Sanitary Engineer, Dorr-Oliver, Inc., Stamford, Conn.

"Sewer Rentals and Sewer Extension Charges in Massachusetts," by Clarence I. Sterling, Jr., Director, Division of Sanitary Engineering, Massachusetts Dept. of Public Health, Boston, Mass.

An extremely interesting round table discussion on "Experiences During the Recent Flood and Lessons Learned Therefrom," was led by J. Henry L. Giles, Engineer of Sanitation, Bureau of Public Works, Hartford, Conn.

Officers elected for 1956 were as follows:

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